THE DESIGN OF

HIGHWAY BRIDGES

AND THE CALCULATION OF STRESSES IN BRIDGE TRUSSES

BY

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PREFACE.

The aim in writing this book has been to give a brief course in the calculation of the stresses in bridge trusses followed by a systematic discussion of the details and the design of highway bridges. This book is supplementary to the author's "The Design of Steel Mill Buildings," and covers the essential part of a first course in bridge design as given in the author's classes.

While there are many excellent books in which the different types of railway bridges are discussed in detail, little attention has heretofore been given to the design of highway bridges. As a consequence of this neglect many of our highway bridges have been very badly designed; the design of these structures being ordinarily left to an engineer without experience, or to the agent of some bridge company who is more interested in the resulting profits than in obtaining a good design. The calculations of the stresses in highway and railway bridges are similar, but the problems in the design of the two types are very different, due to the different requirements and conditions.

In the course in the calculation of stresses both the algebraic and the graphic methods of calculating stresses in bridge trusses are described in detail. While there has been some duplication, in order to make the book self-contained, the calculation of stresses in this book is supplementary to the calculation of stresses in the author's "The Design of Steel Mill Buildings."

Attention is called to the problems in the calculation of stresses in bridge trusses, as given in Chapter IX. It is hoped that the 24 problems given, 20 of which are new, will prove as useful as the list of 22 similar problems given in the author's "The Design of Steel Mill Buildings."

Before taking up the design of bridges the author has had his students calculate the weight, cost and efficiencies of the parts of a highway bridge as given in Part III. This problem makes them familiar with drawings, details and handbooks; and has made it possible to take up the design of bridges without further assistance. For

the details of the design of any part of a truss highway bridge the reader is, therefore, referred to Part III, the problems of investigation and design being essentially the same.

In the discussion of the design and details of highway bridges the author has been guided very largely by his knowledge of the needs of those interested in this subject, gained by an experience in designing and contracting for highway bridges in a dozen states.

The problem of the design of a highway bridge includes the design of both the superstructure and the substructure—all other books on bridges known to the author treat of the superstructure only; and in this book due attention has been given to the design of both superstructure and substructure, and to the effect of the design of the one on the other.

In many locations masonry—stone, concrete or reinforced concrete—bridges are especially suited to the conditions. The design of these structures has been discussed as fully as the limited space permitted.

Cost data are especially valuable to the student in the design of structures, as they show the effect of the different details of design on the cost, and lead to economic designs. The author has discussed in detail the costs of the different parts of highway bridges. These costs are of value principally to the student and to the experienced engineer who is familiar with the conditions of the particular piece of work.

The author is under obligations to many sources to which due credit has been given in the body of the work. The author is also under obligations to many engineers and bridge companies for plans and data; he especially wishes to thank the Boston Bridge Works; the American Bridge Company; the King Bridge Company; Mr. H. A. Fitch, M. Am. Soc. C. E., Chief Engineer, Kansas City Structural Company; Mr. Alex. Maitland, M. Am. Soc. C. E., Chief Engineer, Kansas City Bridge Company; and Mr. F. H. Honens, M. Am. Soc. C. E. Credit is due Professor Howard C. Ford, Iowa State College, formerly Instructor in Civil Engineering in the University of Colorado, for assistance in preparing the drawings.

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UNIVERSITY OF COLORADO,
BOULDER, COLO.
August 23, 1908.

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THE DESIGN OF HIGHWAY BRIDGES.

PART I.

STRESSES IN STEEL BRIDGES.

CHAPTER I.

Types of Bridges.

Introduction.—A truss is a framework composed of individual members so fastened together that loads applied at the joints produce only direct tension or compression. The triangle is the only geometrical figure in which the form is changed only by changing the lengths of the sides. In its simplest form every truss is a triangle or a combination of triangles. The members of the truss are either fastened together with pins, pin-connected, or with plates and rivets, riveted.

The bridge in Fig. 1 consists of two vertical trusses which carry the floor and the load; two horizontal trusses in the planes of the top and bottom chords, respectively, which carry the horizontal wind load along the bridge; and cross-bracing in the plane of the end-posts, called portals, and in the plane of the intermediate posts, called sway bracing. The floor is carried on joists placed parallel to the length of the bridge, and which are supported in turn by the floorbeams. The names of the different parts of the bridge are shown in Fig. 1. The main ties, hip verticals, counters and intermediate posts are together called webs. The bridge shown in Fig. 1 is a through pin-connected bridge of the Pratt type, the traffic passing through the bridge. The bridge shown in Fig. 1 has square abutments; the abutments are not at right angles to the center line of the bridge in a

"skew" bridge. Short span highway and railway bridges have low trusses and no top lateral system nor portals. In a railway bridge the

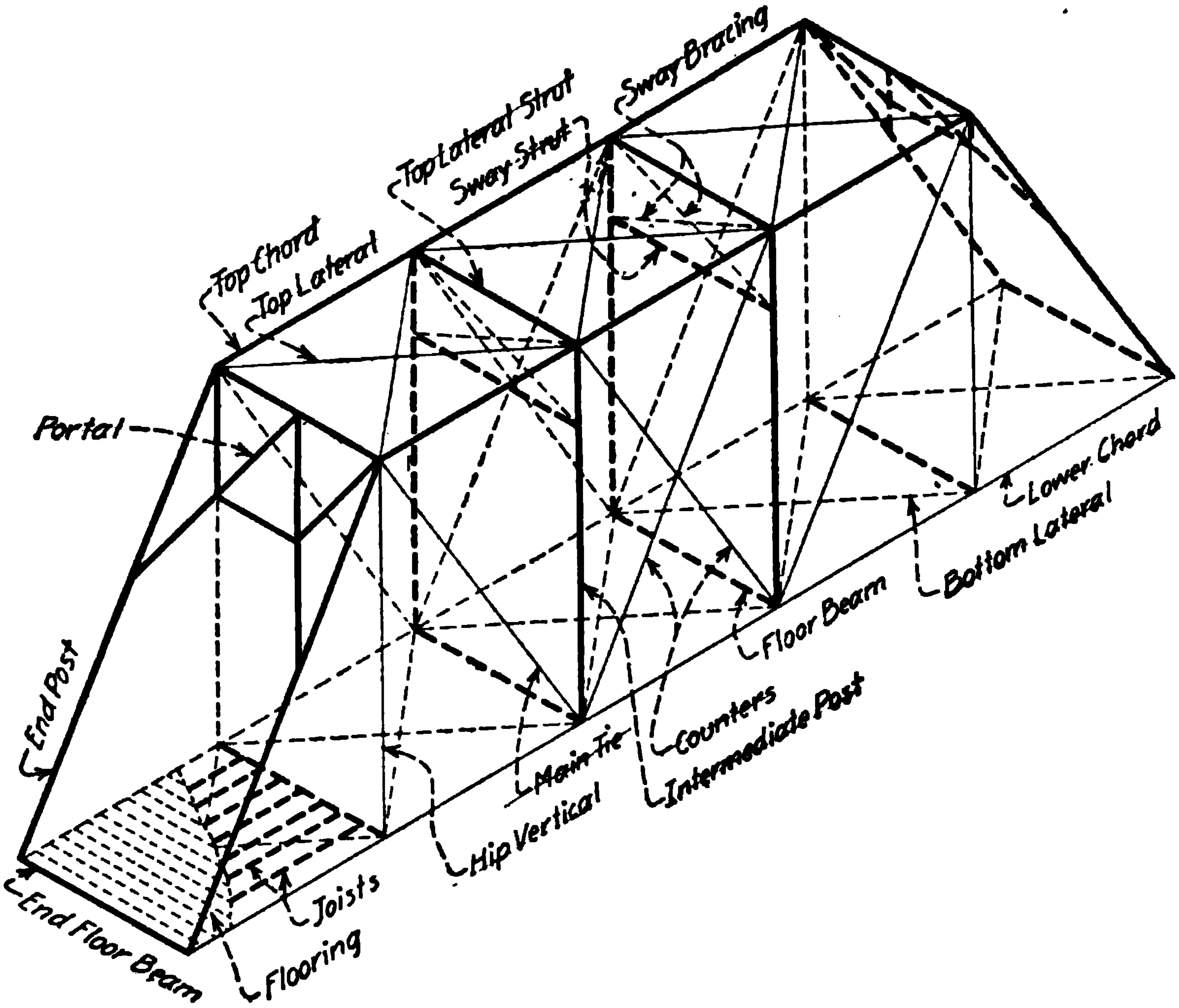


FIG. 1. DIAGRAMMATIC SKETCH OF A THROUGH PRATT TRUSS HIGHWAY BRIDGE.

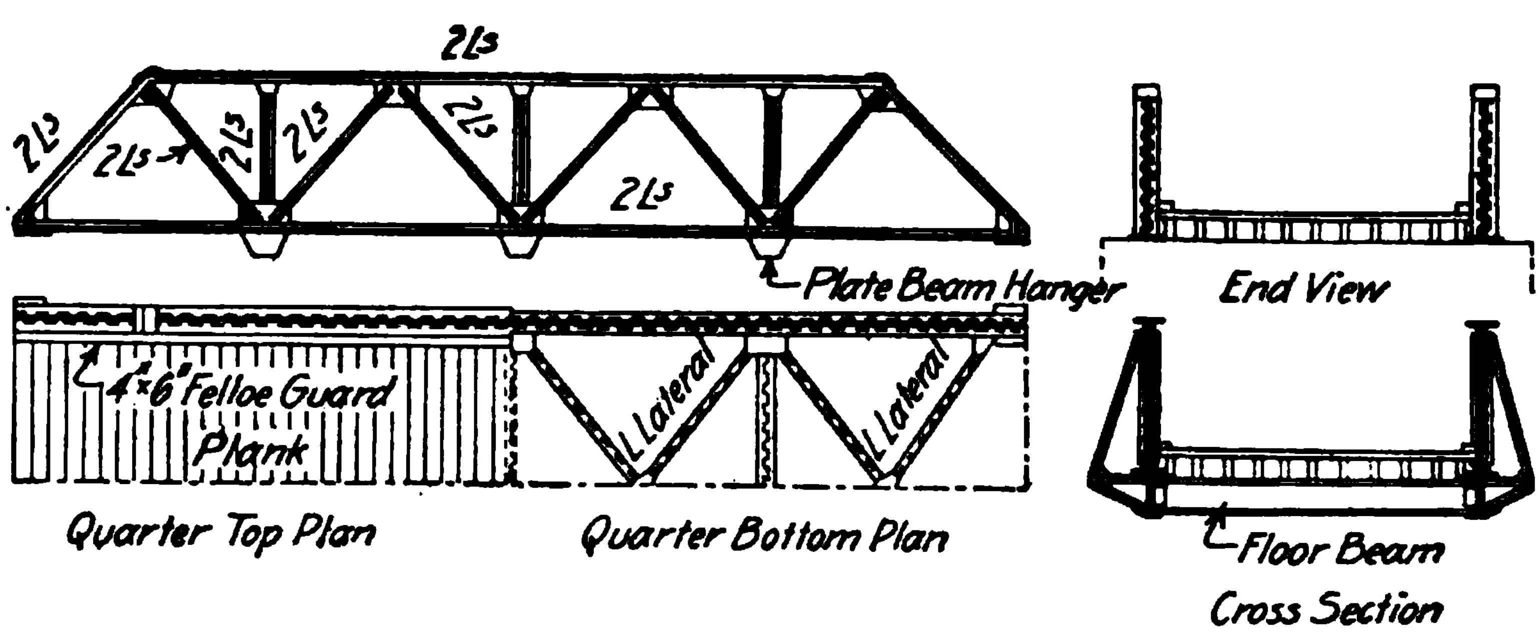


FIG. 2. PLAN OF A LOW OR "PONY" TRUSS HIGHWAY BRIDGE.

track and ties are supported on stringers, which replace the joists in Fig. 1.

Fig. 3. A Warren Low Truss Highway Bridge.

Fig. 4. A Pratt Low Truss Highway Bridge; Seven 100-ft. Spans.

A low truss highway bridge of the Warren type is shown in Fig 2, and a cut of a similar bridge is shown in Fig. 3. The trusses are built up of angles riveted together by means of connection plates. Bridges of

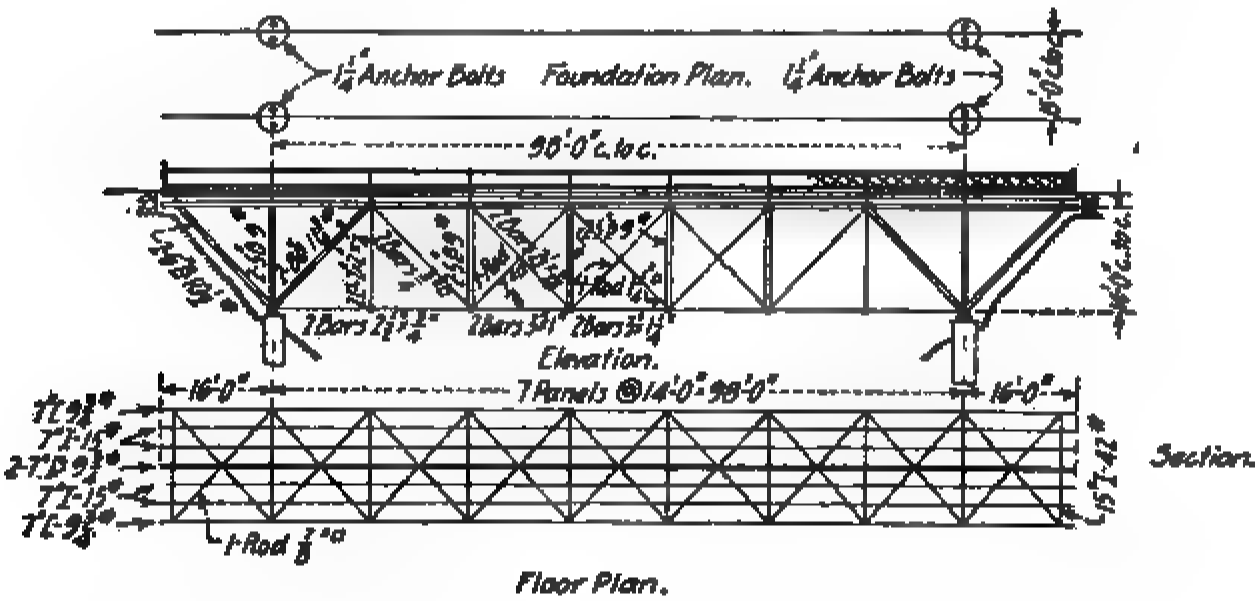


Fig. 5. Deck Pratt Pin-connected Highway Bridge.

Fig. 6. Deck Highway Bridge.

this type are built with spans of from 30 to 75 feet. Low truss bridges are also made with pin-connected joints. A pin-connected low Pratt truss bridge is shown in Fig. 4.

The loads are sometimes carried on the top chord as in Fig. 5, which is a highway bridge built for the U. S. Government in the Yellowstone Park. In this truss the end-posts, top chords and intermediate posts are composed of 2 channels laced; while the lower chords, hip verticals, main ties and counters are composed of eye-bars. The floorbeams are I beams 15 inches deep weighing 50 lbs. per lineal foot (15″ I@50 lbs.), while the joists are 7″ Is and 7″ [s. A deck highway bridge is shown in Fig. 6.

Types of Trusses and Bridges.—The simplest type of bridge is the beam bridge, (*a*) Fig. 7. Beam bridges commonly consist of I beams which span the opening, and are placed near enough together to carry

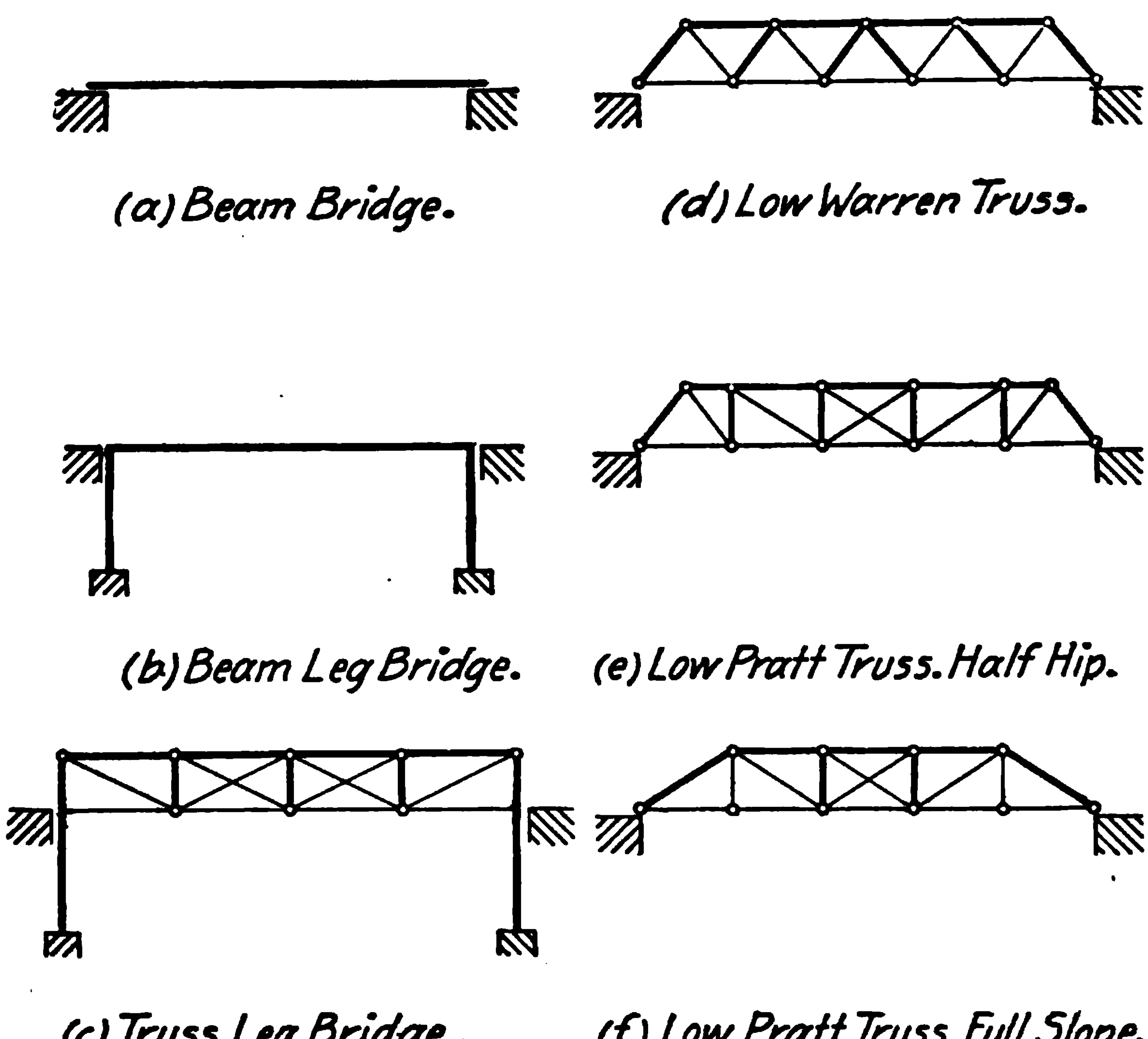


FIG. 7. TYPES OF SHORT SPAN HIGHWAY BRIDGES.

the floor of the bridge. Where foundations are relatively expensive the beams may be carried on posts as in (*b*) Fig. 7. A truss leg bridge

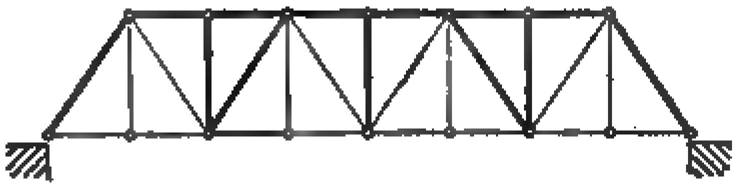


FIG. 8. THROUGH WARREN TRUSS.

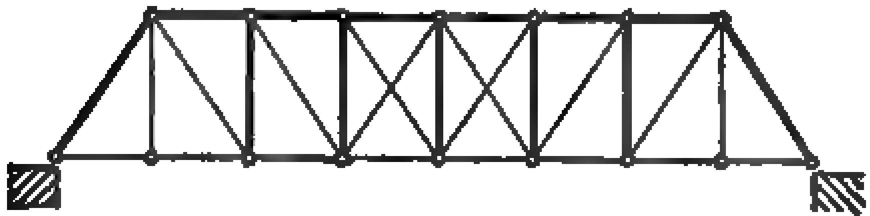


FIG. 9. THROUGH PRATT TRUSS.

is shown in (*c*) Fig. 7. Types (*b*) and (*c*) unless constructed with great care make inferior structures and are not to be recommended.

FIG. 10. THROUGH RIVETED PRATT TRUSS, 111′ 6″ SPAN, OVER ILLINOIS AND MISSISSIPPI CANAL.

A Warren truss is a combination of isosceles triangles as shown in (*d*) Fig. 7 and in Fig. 8. The Pratt truss has its vertical web members in

compression while its diagonal web members are in tension, as shown in (*e*) and (*f*) Fig. 7 and in Figs. 9 and 10. The Warren truss is commonly built with riveted joints while the Pratt truss is usually built with pin-connected joints. The Warren low truss with riveted joints

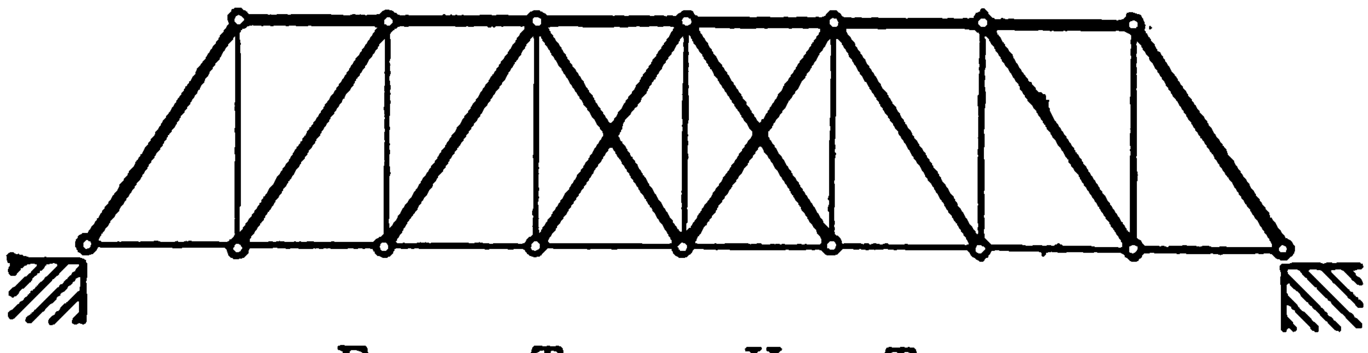


FIG. 11. THROUGH HOWE TRUSS.

as shown in (*d*) is generally preferred in place of the low Pratt truss in either (*e*) or (*f*) Fig. 7. The Howe truss has its vertical web members in tension, and its inclined web members in compression as shown in Fig. 11. The upper and lower chords and the inclined members of a Howe truss are commonly made of timber, while the vertical tension members are iron or steel rods.

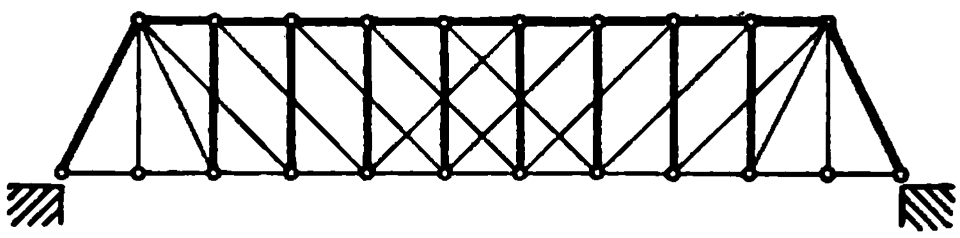


FIG. 12. WHIPPLE TRUSS

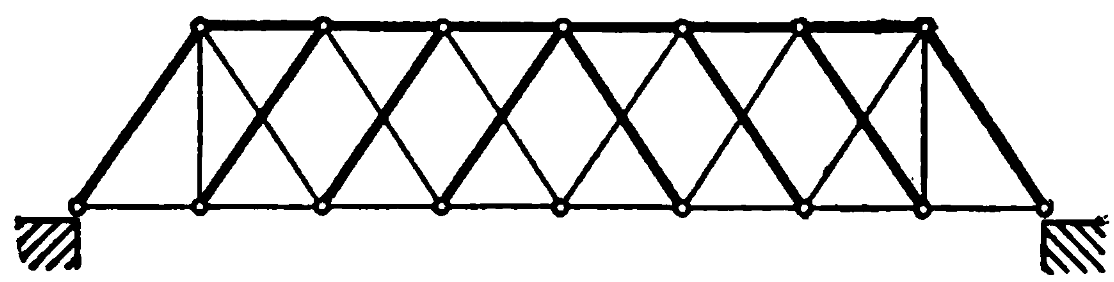


FIG. 13. QUADRANGULAR WARREN TRUSS.

The Whipple truss, Fig. 12, is a double intersection Pratt truss. This truss was designed to give short panels in long spans which have a considerable depth. The stresses in the Whipple truss are indeterminate for moving loads, and its use has been practically abandoned, the Baltimore truss, Fig. 17, being used in its place. The quadrangular Warren truss, Fig. 13 and Fig. 14, with riveted joints, is used as a standard truss for through highway bridges, with spans of from 80 to 170 feet, by the American Bridge Company. Like the Whipple truss its stresses are indeterminate for moving loads.

For spans of from, say, 170 to 240 feet it is quite common to use pin-connected trusses of the Pratt type having inclined chords as in

FIG. 14. THROUGH RIVETED QUADRANGULAR WARREN TRUSS, BUILT BY BOSTON BRIDGE WORKS.

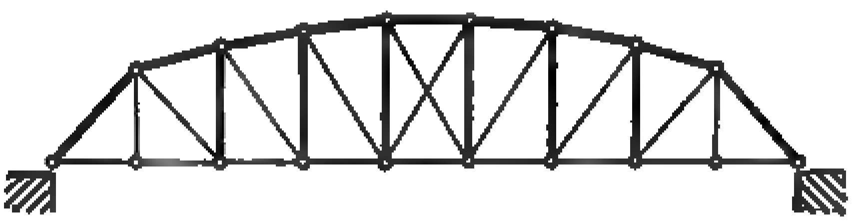


FIG. 15. INCLINED PRATT OR CAMELS-BACK TRUSS.

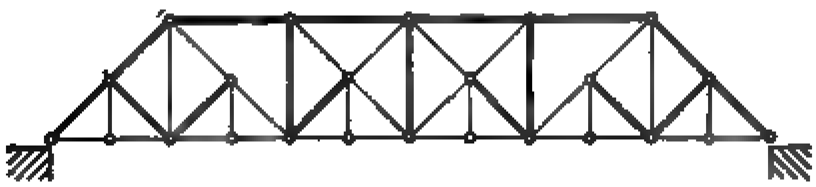


FIG. 17. BALTIMORE TRUSS.

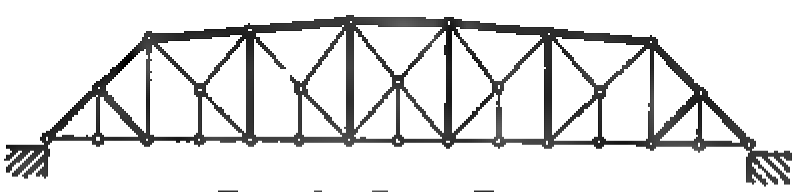


FIG. 18. PETIT TRUSS.

Fig. 15 and Fig. 16. Where the truss has an even number of panels the author has subdivided the center panel as shown in Fig. 20.

The Baltimore truss, Fig. 17, is a Pratt truss with parallel chords

FIG. 16. PARKER OR CAMELS-BACK, PIN-CONNECTED HIGHWAY BRIDGE, BUILT BY AMERICAN BRIDGE COMPANY.

in which the main panels have been subdivided by an auxiliary framework. The auxiliary framework may have struts as in Fig. 17, or ties as in Fig. 18. The Baltimore truss with inclined upper chords,

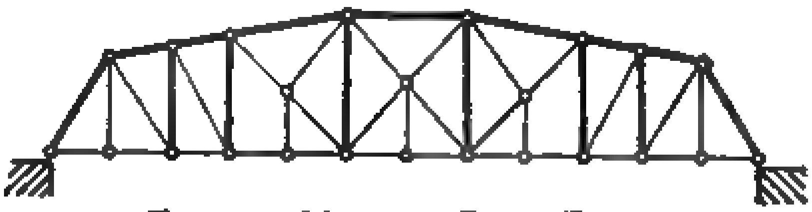


FIG. 19. MODIFIED PETIT TRUSS.

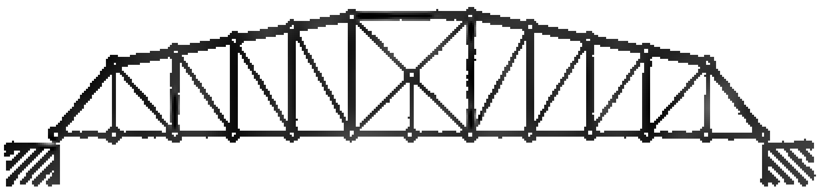


FIG. 20. MODIFIED CAMELS-BACK TRUSS.

FIG. 21. PLATE GIRDER HIGHWAY BRIDGE, BUILT BY AMERICAN BRIDGE COMPANY.

Fig. 18, is called a Petit truss. A modified form of the Petit truss that is sometimes used, is shown in Fig. 19. The stresses in Baltimore and Petit trusses are statically determinate for all conditions of loading. These trusses are economical in construction and satisfactory in ser-

vice, and have almost entirely replaced the Whipple truss for long span bridges.

The types of simple bridge trusses described above are those that are in the most common use, although quite a number of other types of trusses have been used and abandoned.

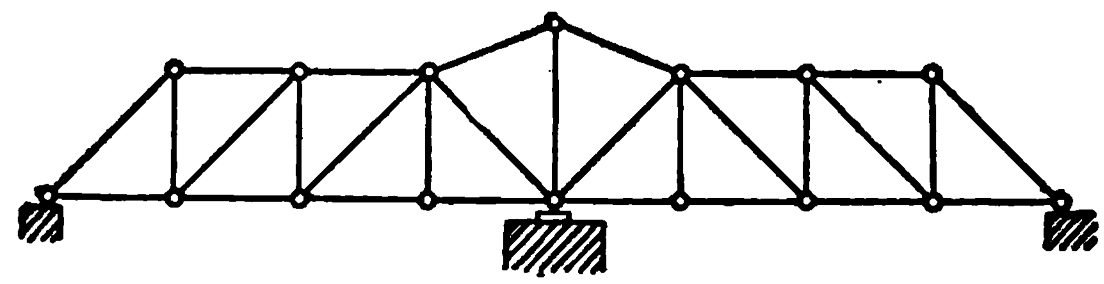


FIG. 22. SWING BRIDGE, CENTER BEARING.

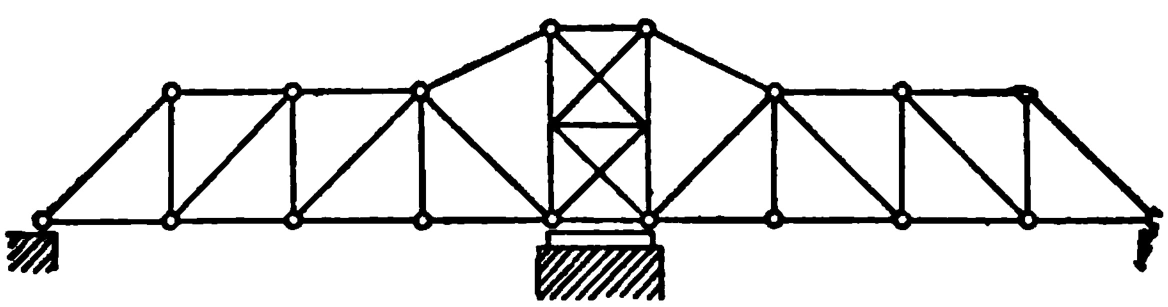


FIG. 23. SWING BRIDGE, TURNTABLE BEARING.

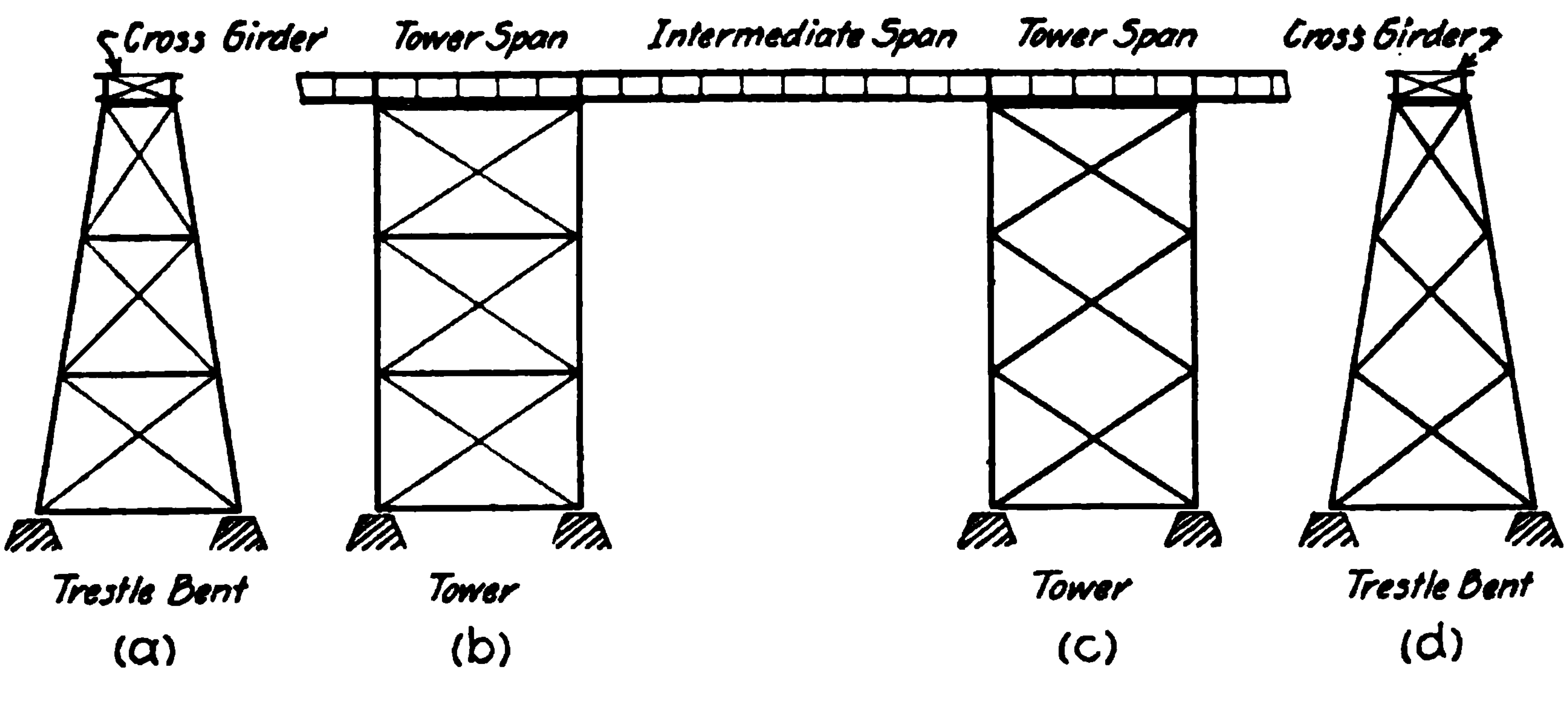


FIG. 24. RAILWAY STEEL TRESTLE.

BEAMS AND PLATE GIRDERS.—For spans of, say, 30 feet and under rolled beams are often used to carry the roadway, while for spans from about 30 to 100 feet plate girders are used. When the roadway is carried on top of the girders, the bridge is called a deck plate girder bridge, and when the roadway passes between the girders, the bridge is called a through plate girder bridge as in Fig. 21.

SWING BRIDGES.—Swing bridges may be made of plate girders or trusses, and may turn on a center pivot as in Fig. 22, or on a turntable supported on a drum as in Fig. 23. The center pivot swing bridge has two spans continuous over the pivot support, while the turntable swing bridge has three spans ordinarily continuous over the two turntable supports. When the swing bridge is open each arm acts as a simple cantilever span.

STEEL TRESTLES.—Steel trestles are used for carrying the roadway at a considerable distance above the ground, Fig. 24. The tower and intermediate spans are commonly built of plate girders, whether the trestle carries a railroad or a highway roadway. The towers consist of two trestle bents as in (*a*) or (*d*), braced together by longitudinal bracing as in (*b*) or (*c*) Fig. 24. Bracing as in (*a*) and (*b*) is used with either adjustable or rigid diagonal members, while bracing (*c*) and (*d*) is used only for rigid members.

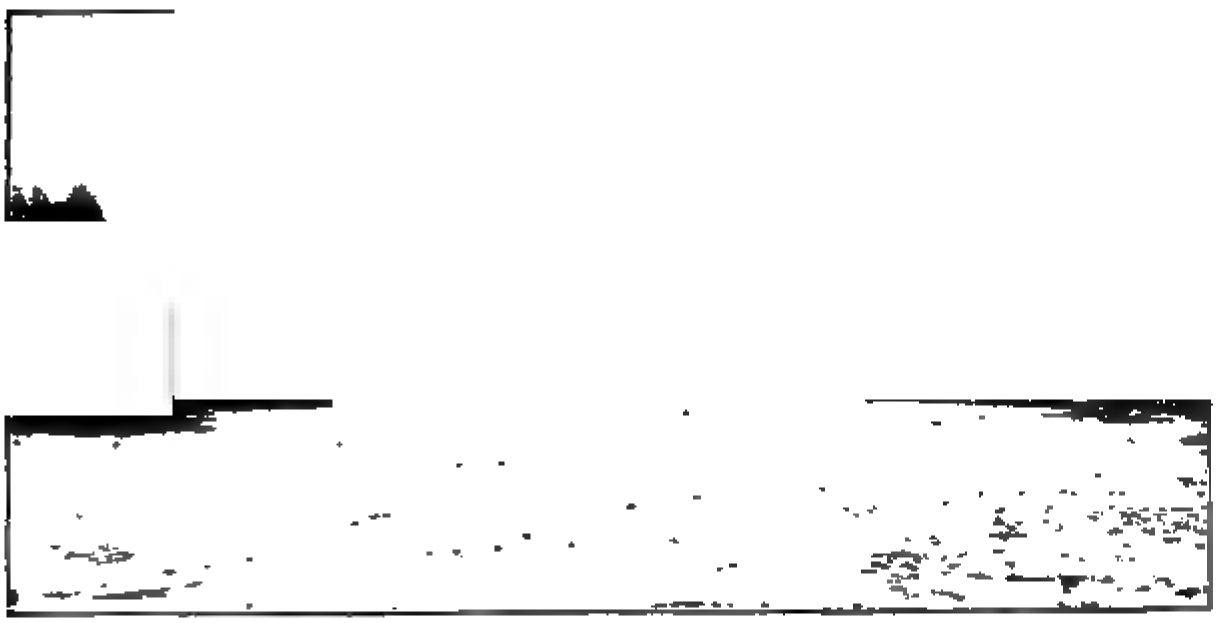


FIG. 25. CLARITON-CLIFTON TWO-HINGED ARCH HIGHWAY BRIDGE OVER NIAGARA RIVER.

STEEL ARCHES.—Steel arch bridges are made (1) with three hinges, (2) with two hinges, and (3) without hinges, and may have solid webs, or spandrel or open webs. A two-hinged highway arch is shown in Fig. 25.

FIG. 26. CANTILEVER HIGHWAY BRIDGE.

FIG. 27. SUSPENSION HIGHWAY BRIDGE OVER NIAGARA RIVER AT QUEENSTOWN, ONTARIO.

CANTILEVER BRIDGES.—A cantilever bridge consists of two anchor spans, which support a suspended or channel span. The shore ends of the anchor spans are anchored to the shore pier and are sup-

ported on the river pier. A cantilever highway bridge is shown in Fig. 26.

SUSPENSION BRIDGES.—In a suspension bridge the roadway is supported by hangers attached to the main cables. Stiffening trusses are placed above the plane of the roadway to assist in distributing the live loads and for the purpose of increasing the rigidity of the structure. The suspension highway bridge over the Niagara River at Queenstown, Ont., is shown in Fig. 27.

Simple truss bridges, beam and plate girder bridges, only, will be considered in this book.

CHAPTER II.

LOADS AND WEIGHTS OF HIGHWAY BRIDGES.

LOADS.—The loads carried by a bridge consist of (1) fixed or dead loads, (2) a moving or live load, and (3) miscellaneous loads. The dead load consists of the weight of the structure and is always

TABLE I.

SUMMARY OF WEIGHT OF METAL.

111′ 6″ × 18′ 0″ RIVETED HIGHWAY BRIDGE.

Ref. No.	Member.	Weights.			Details Per Cent. of Main Members.
		Main Members.	Details.	Total.	
1	End Posts.	5,592	3,892	9,484	67.0
2	Top Chords.	5,900	3,942	9,842	67.0
3	Lower Chords.	5,232	442	5,674	8.5
4	Intermediate Posts.	2,436	2,835	5,277	116.0
5	Main Ties.	3,184	474	3,658	15.0
6	Hip Verticals.	856	163	1,019	19.0
7	Counters.	1,156	109	1,265	9.0
8	Floorbeams.	8,350	2,230	10,580	27.0
12	Struts.	1,486	544	2,030	36.0
13	Top Laterals.	531	35	566	7.0
14	Bottom Laterals.	843	182	1,025	21.0
15	Portals.	1,732	620	2,352	36.0
16	Pins and Nuts.		86	86	
17	Pedestals.		1,949	1,949	
		37,298	17,503	54,801	46.9

Total Weight of Metal in Bridge, exclusive of 9, 10, 11, 18 and 19 = 54,801 lbs.

Ref. No.	Member.	Main Members.	Details.	Total.	Details Per Cent. of Main Members.
9	Joists.	23,852	2,200	26,052	9.0
10	Hub Guard.	2,392	267	2,659	11.0
11	End Struts.	469	167	636	36.0
18	Bolts for Lumber.		365	365	
19	Spikes for Lumber.		389	389	
		26,713	3,388	30,101	13.0
Total Metal in Bridge.		64,011	20,891	84,902	33.0

carried by the bridge; the live load consists of the moving load which the bridge is built to carry; while the miscellaneous loads include wind loads, snow loads, etc.

Weight of Bridges.—The weight of a bridge is composed of (1) the weight of the steel in the steel framework, consisting of the vertical trusses, the upper and lower lateral systems, the floorbeams,

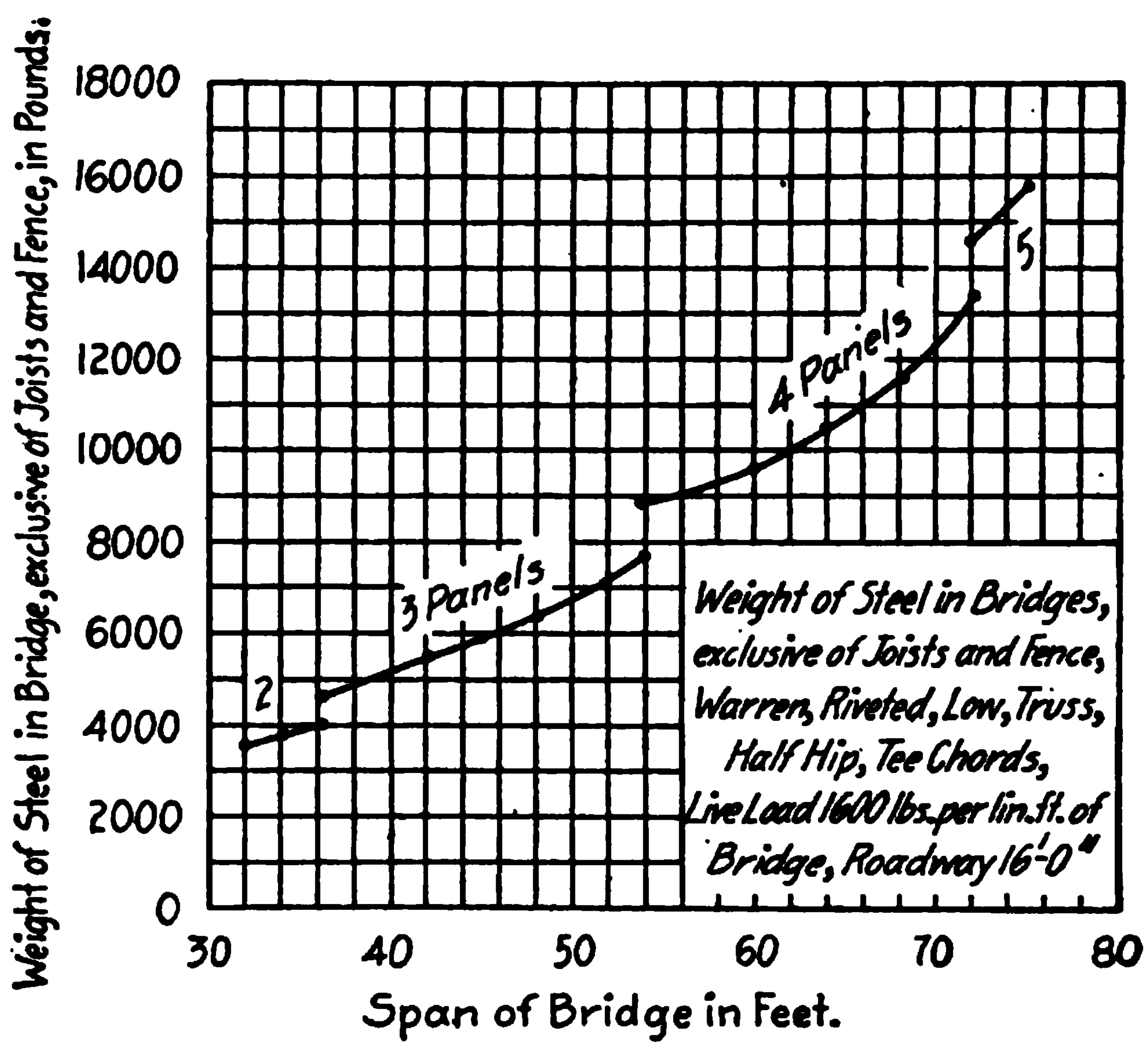


FIG. 29. TOTAL WEIGHT OF STEEL IN WARREN, RIVETED, LOW TRUSS HIGHWAY BRIDGES, EXCLUSIVE OF JOISTS AND FENCE. (GILLETTE-HERZOG MFG. CO.)

the portals and sway bracing; (2) the weight of the joists and the fence; and (3) the weight of the floor covering.

Weight of Steel Framework.—The weight of the steel framework in a bridge will depend upon whether it is a railway or a highway bridge, upon the span and the type of bridge, and upon the load which it has been designed to carry. The weight of the steel in a highway bridge is the sum of the weights of the parts (1), (2) and (3) of the bridge as shown in Table I.

WEIGHT OF STEEL HIGHWAY TRUSS BRIDGES. Gillette-Herzog Mfg. Co. Standards.—The total weights of the steel in highway bridges without sidewalks, exclusive of joists, fence, etc., are given in Fig. 29, Fig. 30 and Fig. 31. These bridges were designed for the loadings stated and all have a roadway of 16 feet except the Petit trusses with spans above 228 feet, which have an 18-ft. roadway. The bridges were designed by the Gillette-Herzog Mfg. Co. and are excellent structures.

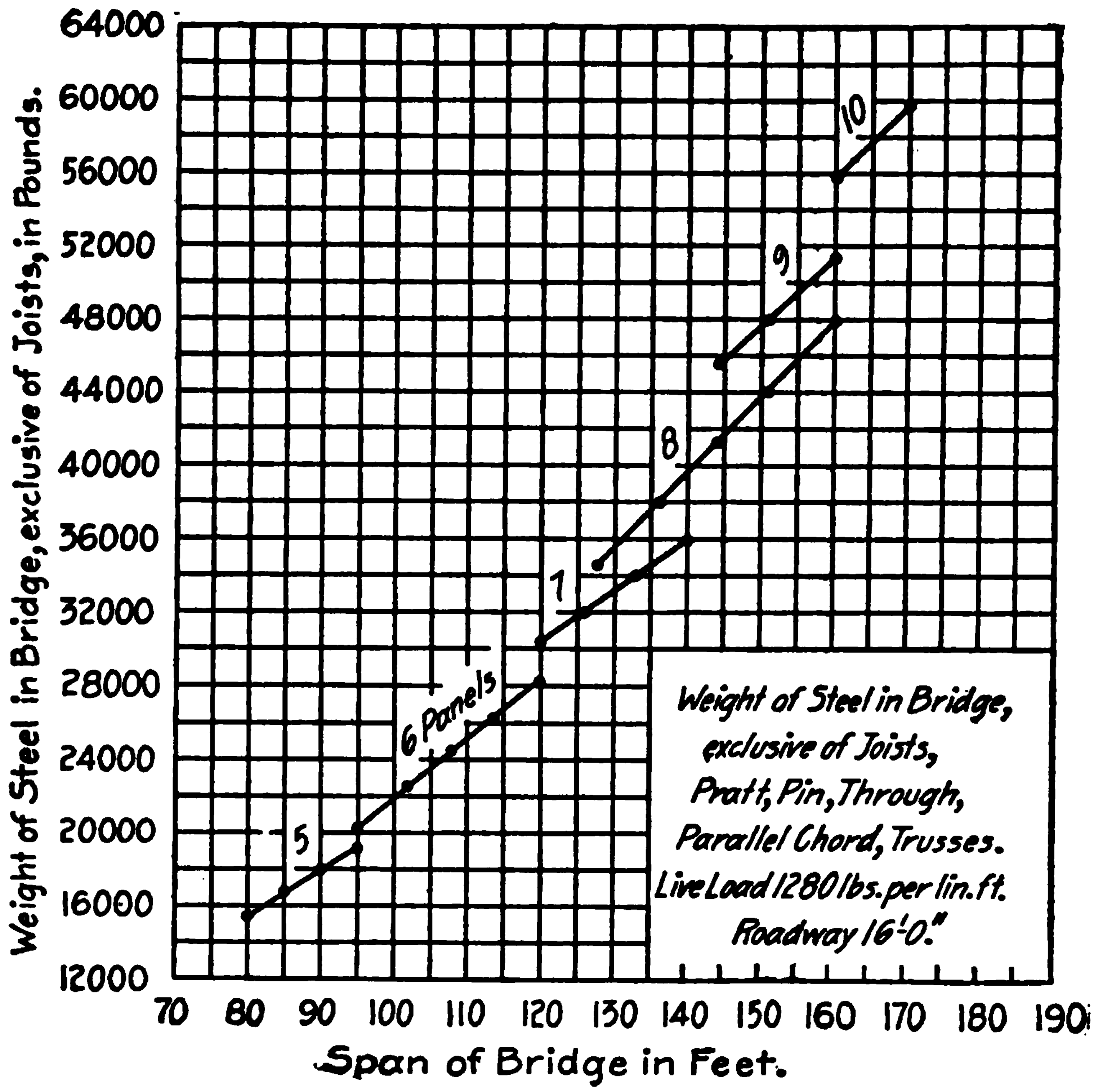


FIG. 30. TOTAL WEIGHT OF STEEL IN PRATT, PIN-CONNECTED, THROUGH PARALLEL CHORD HIGHWAY BRIDGES, EXCLUSIVE OF JOISTS AND FENCE. (GILLETTE-HERZOG MFG. CO.)

The effect of panel length on the weight of bridges is plainly shown in Fig. 30. By continuing the lines the weights of the bridge for different numbers of panels may be read directly from the diagram.

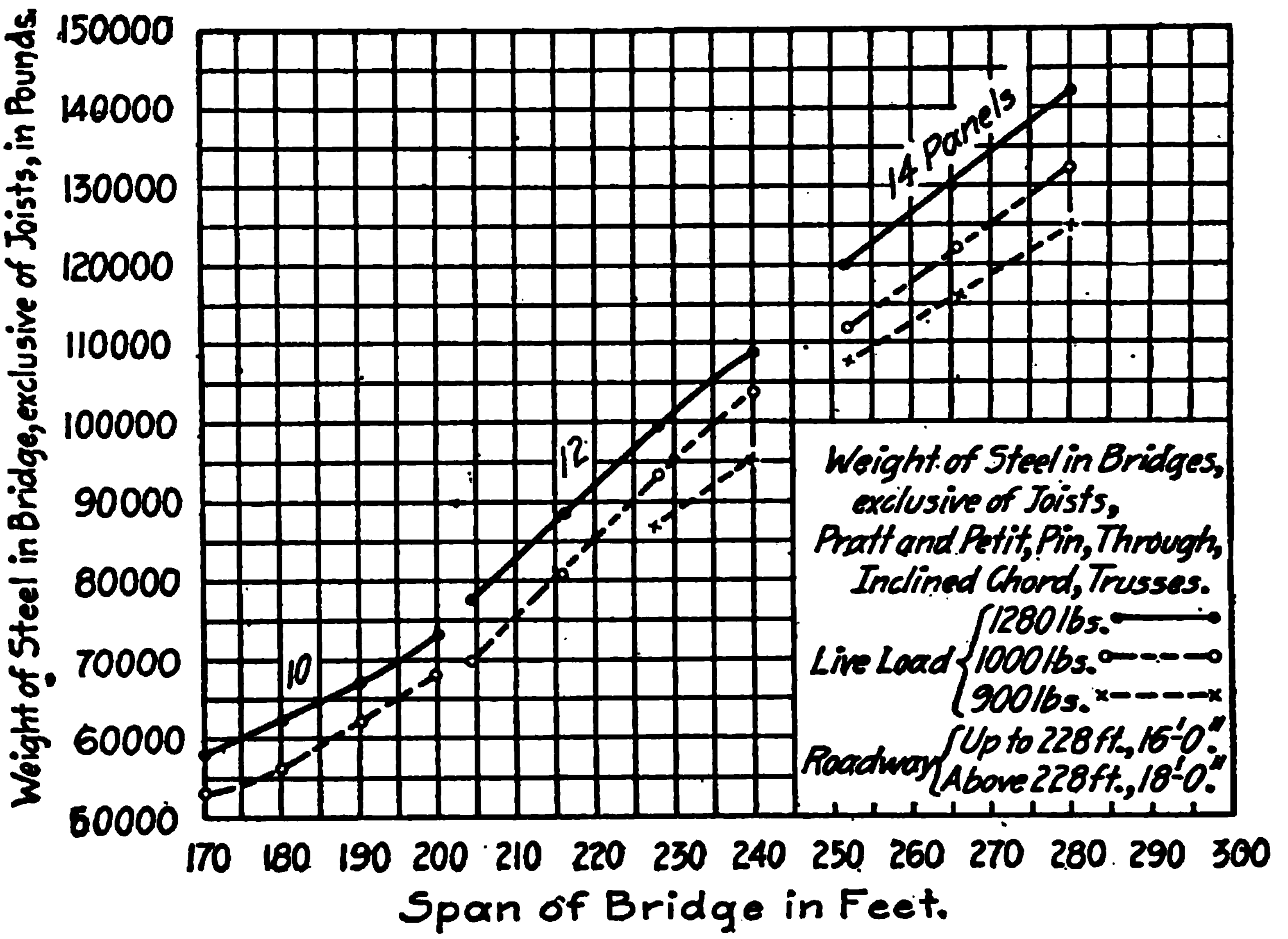


FIG. 31. TOTAL WEIGHT OF STEEL IN PRATT AND PETIT THROUGH INCLINED CHORD HIGHWAY BRIDGES, EXCLUSIVE OF JOISTS AND FENCE. (GILLETTE-HERZOG MFG. CO.)

Boston Bridge Works Standards.—The total weights of steel, exclusive of joists and fence, in highway bridges as designed by the Boston Bridge Works, are given in Figs. 32 and 33. The bridges whose weights are given in Fig. 32 were designed for a live load of 80 lbs. per square foot for the trusses, and 100 lbs. per square foot for the floor system. The bridges whose weights are given in Fig. 33 were designed for a live load of 80 lbs. per square foot for the trusses, and 100 lbs. per square foot for the floor system of the main roadway and the footwalks. The bridges were designed with allowable tension of 15,000 lbs. per square inch in main members and 18,000 lbs. per square inch in laterals; allowable compression of 12,000 and 15,000 lbs. per square inch, reduced by Rankine's column formula, in main members and laterals, respectively.

Weights of Trusses, Laterals, etc.—The weights of one truss of steel highway bridges as designed for different loadings by the Boston

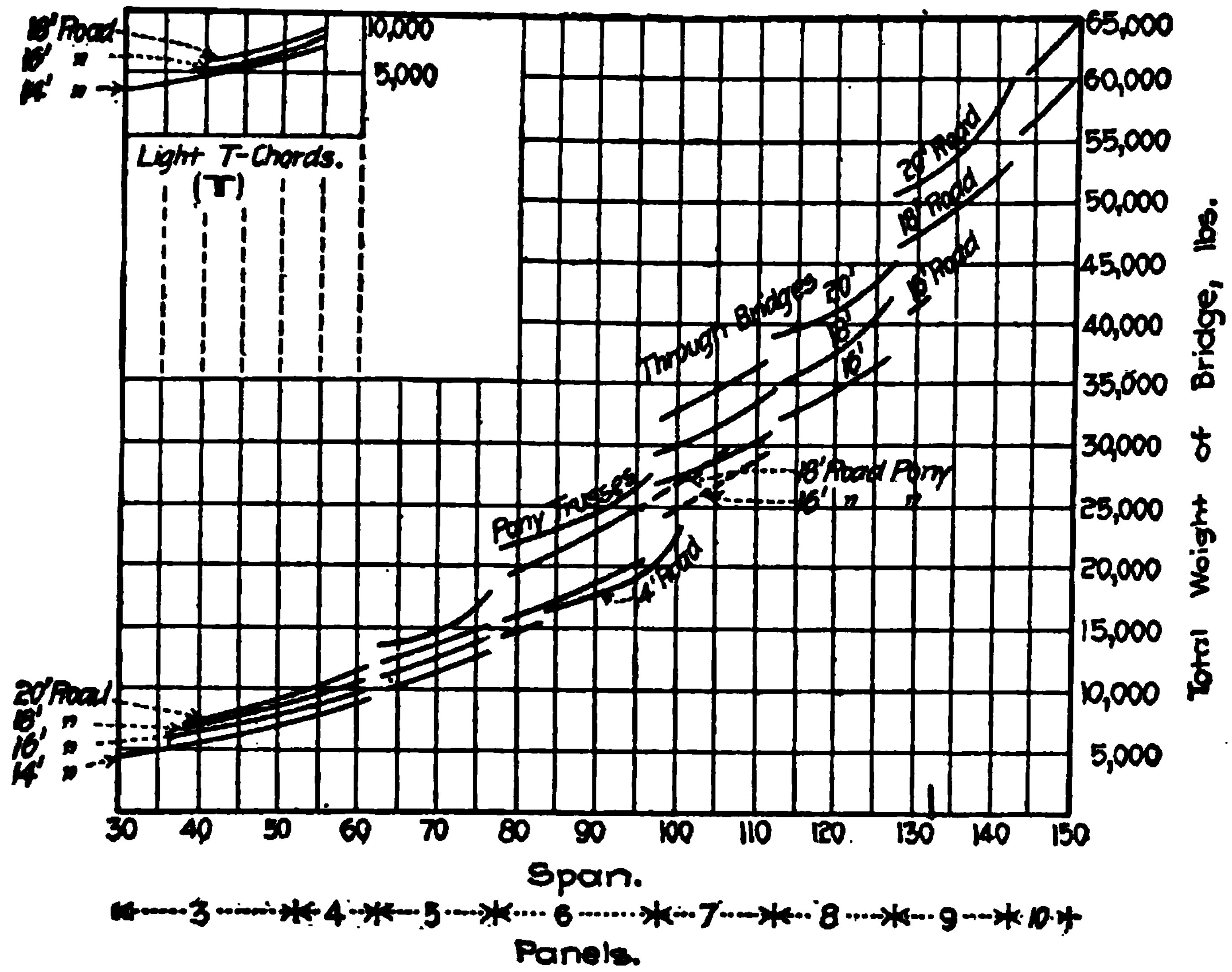


FIG. 32. TOTAL WEIGHT OF STEEL IN HIGHWAY BRIDGES, EXCLUSIVE OF JOISTS AND FENCE. (BOSTON BRIDGE WORKS.)

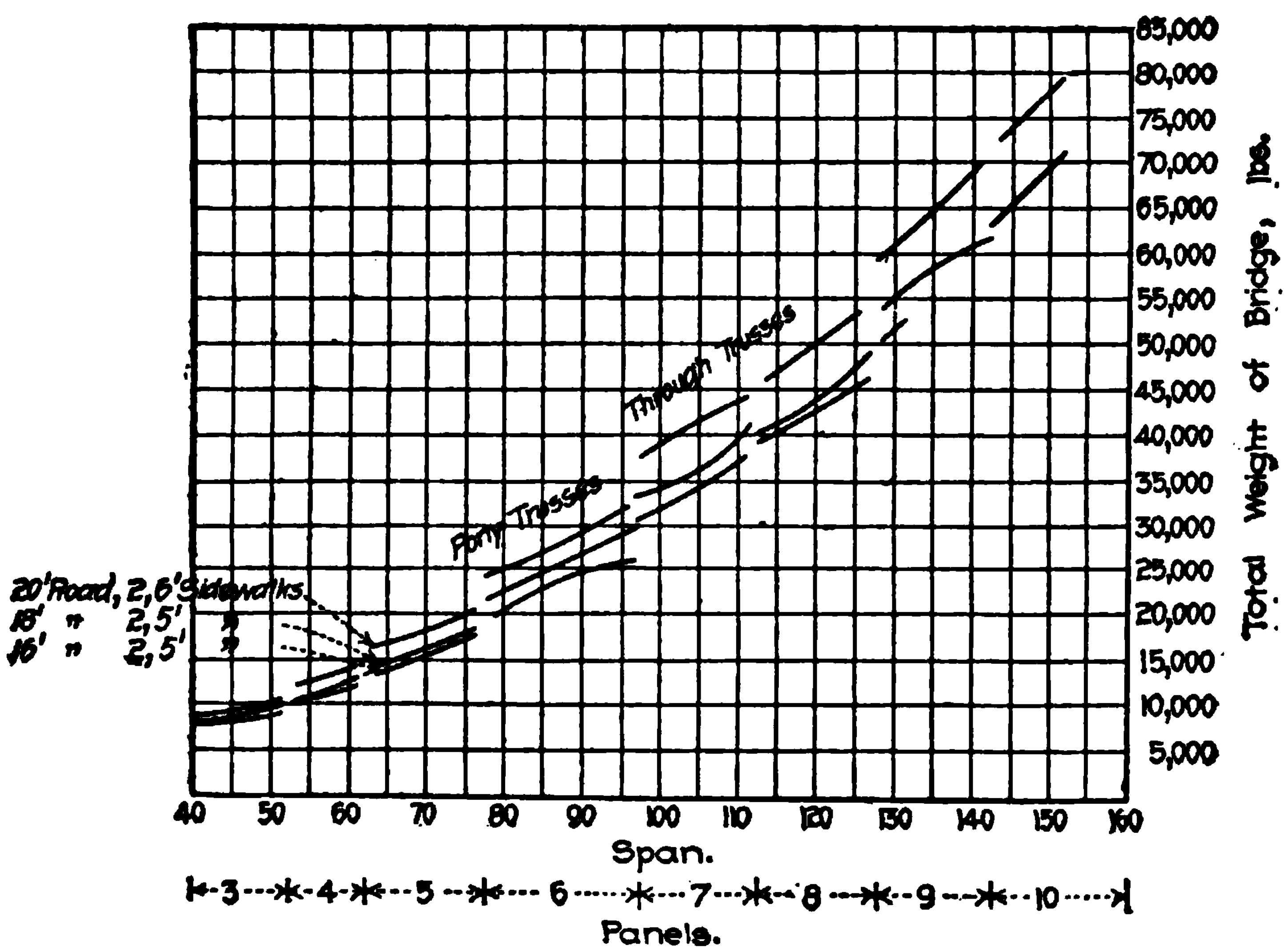


FIG. 33. TOTAL WEIGHT OF STEEL IN HIGHWAY BRIDGES WITH SIDEWALKS, EXCLUSIVE OF JOISTS AND FENCE. (BOSTON BRIDGE WORKS.)

Bridge Works, are given in Table II. The live and dead loads are in pounds per lineal foot per truss. Class I was designed for a live load of 1,280 lbs. per lineal foot of bridge; Class II for 3,600 lbs. per lineal foot of bridge; Class III for 6,000 lbs. per lineal foot of bridge. The trusses given in Table II were designed for a tension of 15,000 and 18,000 lbs. per square inch in main members and laterals, respectively;

TABLE II.

WEIGHT OF ONE TRUSS OF A STEEL PRATT HIGHWAY TRUSS BRIDGE. DEAD AND LIVE LOADS IN POUNDS PER LINEAL FOOT OF TRUSS. (BOSTON BRIDGE WORKS.)

Span.	Class I.			Class II.			Class III.		
	Live Load.	Dead Load.	Weight of Truss, lbs.	Live Load.	Dead Load.	Weight of Truss, lbs.	Live Load.	Dead Load.	Weight of Truss, lbs.
40	640	250	1,700	1,800	520	2,500	3,000	840	3,500
45		255	2,000		530	3,100		855	4,300
50		260	2,300		540	3,800		870	5,300
55		265	2,700		550	4,600		885	6,300
60		270	3,200		560	5,400		900	7,600
65		275	3,700		570	6,400		915	9,000
70		280	4,200		580	7,500		930	10,500
75		285	4,800		590	8,600		945	12,200
80		290	5,400		600	9,900		960	14,000
85		295	6,100		610	11,200		975	15,900
90		300	6,800		620	12,600		990	18,100
95		305	7,500		630	14,100		1,005	20,300
100		310	8,300		640	15,600		1,020	22,700
105		315	9,100		650	17,300		1,035	25,200
110		320	10,000		660	19,100		1,050	27,900
115		325	10,900		670	21,000		1,065	30,700
120		330	11,900		680	23,000		1,080	33,700
125		335	12,900		690	24,900		1,095	36,800
130		340	14,000		700	27,100		1,110	40,100
135		345	15,100		710	29,300		1,125	43,500
140		350	16,300		720	31,600		1,140	47,100
145		355	17,500		730	34,000		1,155	50,800
150		360	18,700		740	36,500		1,170	54,600
155		365	20,000		750	39,100		1,185	58,600
160		370	21,300		760	41,700		1,200	62,700
165		375	22,700		770	44,500		1,215	67,000
170		380	24,100		780	47,400		1,230	71,400
175		385	25,600		790	50,300		1,245	76,000
180		390	27,100		800	53,300		1,260	80,700
185		395	28,700		810	56,400		1,275	85,600
190		400	30,300		820	59,700		1,290	90,600
195		405	32,000		830	63,000		1,305	95,700
200		410	33,600		840	66,400		1,320	101,000

a compression of 12,000 and 15,000 lbs. per square inch, reduced by Rankine's formula, in main members and laterals, respectively; and for rivet shear of 10,500 lbs. per square inch, and rivet bearing of 15,000 lbs. per square inch.

The weights of one floorbeam, designed for different loadings per lineal foot of floorbeam, are given in Table III.

TABLE III.

WEIGHT OF ONE FLOORBEAM IN POUNDS FOR DIFFERENT LOADS PER LINEAL FOOT OF FLOORBEAM. (BOSTON BRIDGE WORKS.)

Width of Roadway. Feet.	Total Load on One Floorbeam.					
	900	1,300	1,700	2,100	2,500	2,900
12	330	390	460	510	550	
16	630	710	830	930	1,020	
20	930	1,050	1,200	1,370	1,430	
24	1,260	1,430	1,620	1,850	2,080	
28	1,630	1,860	2,100	2,380	2,700	
32	2,050	2,380	2,700	3,080	3,400	
40	2,994	3,863	4,418	4,922	5,581	6,200

TABLE IV.

WEIGHT OF SIDEWALK BRACKETS IN POUNDS. (BOSTON BRIDGE WORKS.)

Width of Walk, Feet.	Total Load on Bracket.				
	900	1,300	1,700	2,100	2,500
5	90	115	140	160	185
6	135	155	180	205	230
7	175	200	225	255	285
8	215	250	280	315	360
9	260	295	340	390	450
10	300	355	400	465	560

The weights of one sidewalk bracket, designed for a live load of 100 lbs. per square foot, for different widths of sidewalk, are given in Table IV.

The weights of shoes and rollers, for one span for reactions of 50,000 lbs. to 400,000 lbs., are given in Table V. The bearing on masonry was taken at 300 lbs. per square inch, while the bearing on rollers was taken as $900\sqrt{D}$ lbs. per lineal inch of roller, where D is the diameter of the roller.

TABLE V.

WEIGHT OF SHOE PLATES (PEDESTALS) AND ROLLERS IN POUNDS FOR ONE SPAN. (BOSTON BRIDGE WORKS.)

REACTION, POUNDS.	WEIGHT, POUNDS.	REACTION, POUNDS.	WEIGHT, POUNDS.
50,000	280 + 240*	250,000	2,080 + 1,520*
100,000	620 + 450	300,000	2,660 + 2,100
150,000	1,030 + 850	350,000	3,300 + 2,450
200,000	1,540 + 1,220	400,000	3,940 + 2,800

* Amount to add for 2 cast shoes equal in thickness to rollers and bottom plate.

The weights of lateral bracing for different spans and roadways are given in Table VI. The wind load on the unloaded chord was taken as 150 lbs. per lineal foot of bridge, while the wind load on loaded chord was taken as 300 lbs. per lineal foot of bridge, of which 150 lbs. was considered a moving load.

It will be seen in Figs. 29 to 33 that the weights of highway

TABLE VI.

WEIGHT OF LATERAL BRACING IN POUNDS FOR STEEL HIGHWAY BRIDGES. (BOSTON BRIDGE WORKS.)

SPAN FT.	SIXTEEN FT. ROADWAY.				TWENTY-FOUR FT. ROADWAY.				THIRTY-TWO FT. ROADWAY.			
	Top.	Bottom.	Portal.	Total.	Top.	Bottom.	Portal.	Total.	Top.	Bottom.	Portal.	Total.
40		846				786				933		
50		1,000				1,000				1,200		
60		1,160				1,300				1,500		
70		1,350				1,650				1,850		
80	950	1,550	600	3,100	2,200	1,950	1,400	5,550	3,300	2,150	1,550	7,000
90	1,100	1,750	750	3,600	2,600	2,250	1,600	6,450	3,850	2,500	1,900	8,150
100	1,200	2,000	950	4,150	3,000	2,550	1,850	7,400	4,300	2,800	2,250	9,350
110	1,400	2,250	1,100	4,750	3,400	2,950	2,050	8,400	4,750	3,150	2,650	10,550
120	1,602	2,559	1,336	5,497	3,880	3,323	2,392	9,595	5,183	3,566	3,066	11,815
130	1,750	2,800	1,500	6,050	4,200	3,650	2,550	10,400	5,500	3,850	3,450	12,800
140	1,950	3,150	1,700	6,800	4,550	4,000	2,850	11,400	5,850	4,150	3,850	13,850
150	2,150	3,600	1,950	7,700	4,950	4,400	3,100	12,450	6,200	4,450	4,350	15,000
160	2,400	4,050	2,150	8,600	5,300	4,800	3,350	13,450	6,500	4,750	4,800	16,050
170	2,650	4,500	2,450	9,600	5,650	5,150	3,650	14,450	6,850	5,050	5,350	17,250
180	2,900	5,000	2,700	10,600	6,000	5,600	4,000	15,600	7,150	5,350	5,850	18,350
190	3,200	5,500	3,000	11,700	6,350	6,000	4,300	16,650	7,450	5,650	6,400	19,500
200	3,544	6,094	3,388	13,026	6,653	6,495	4,652	17,800	7,707	5,917	7,016	20,640

bridges for any given span depends upon (1) the type of truss, (2) the loading, (3) the number of panels in the bridge, and (4) upon the width of the roadway.

American Bridge Company Standards.—To cover a larger range of conditions the author has tabulated graphically the weights of two trusses of low Pratt pin-connected bridges in Fig. 34; in Fig. 35 he has tabulated the weights of one floorbeam for various panel lengths and widths of roadway for a live load of 100 lbs. per square foot of

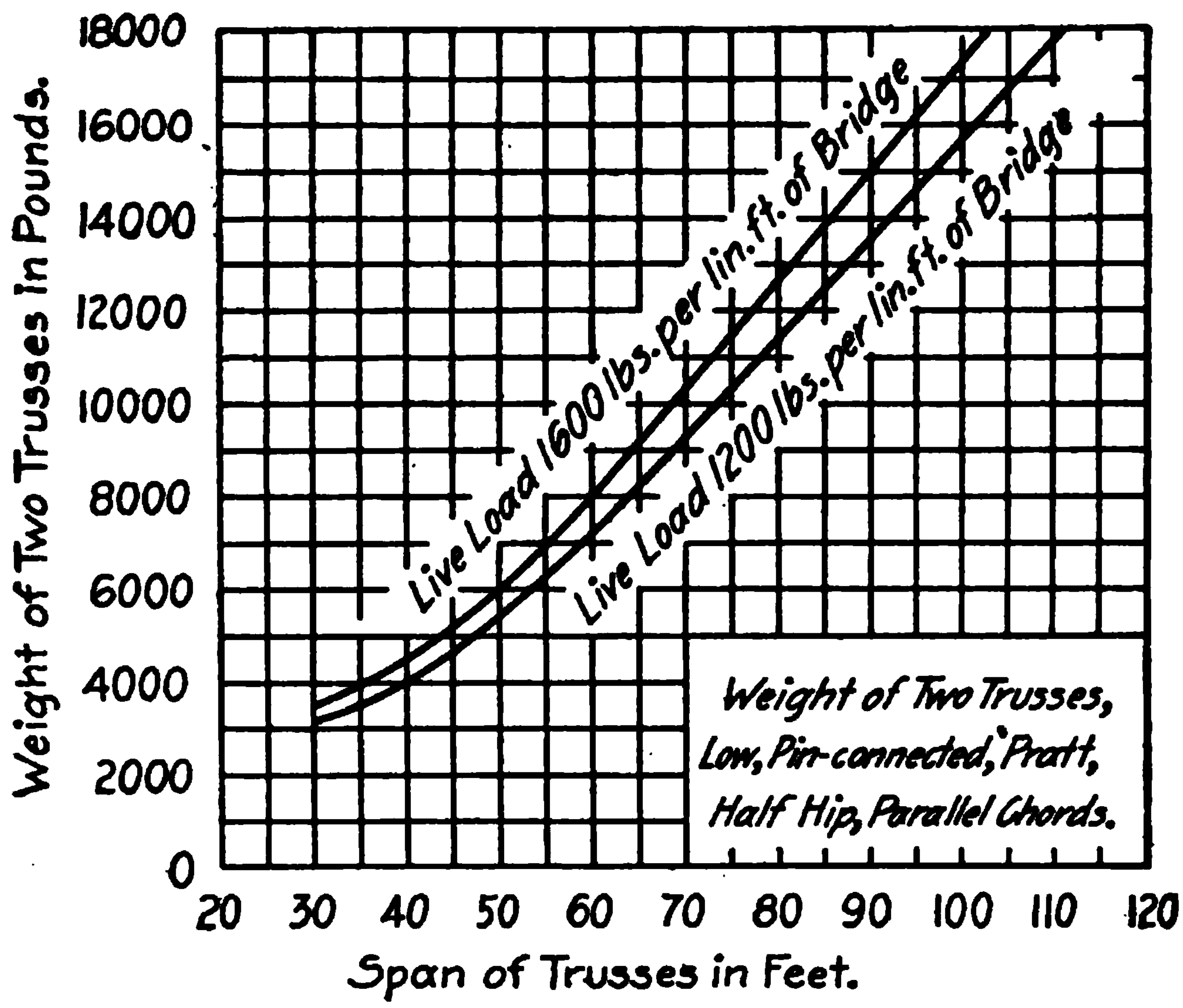


FIG. 34. TOTAL WEIGHT OF STEEL IN TWO TRUSSES OF A LOW, PIN-CONNECTED, HALF HIP, PRATT HIGHWAY BRIDGE (FIG. 7). (AMERICAN BRIDGE CO.)

roadway; while in Fig. 36 he has tabulated the weights of lateral systems, including portals per foot of clear width of the bridge. The upper lateral systems were designed for 150 lbs. per lineal foot of bridge, and the lower lateral systems were designed for a wind load of 300 lbs. per lineal foot of bridge, 150 lbs. being considered a moving load. For example to find the weight of a five panel, low, Pratt, pin-connected highway bridge having a span of 80 feet and a clear roadway of 18 feet, the trusses being designed for a live load of 1,600 lbs. per

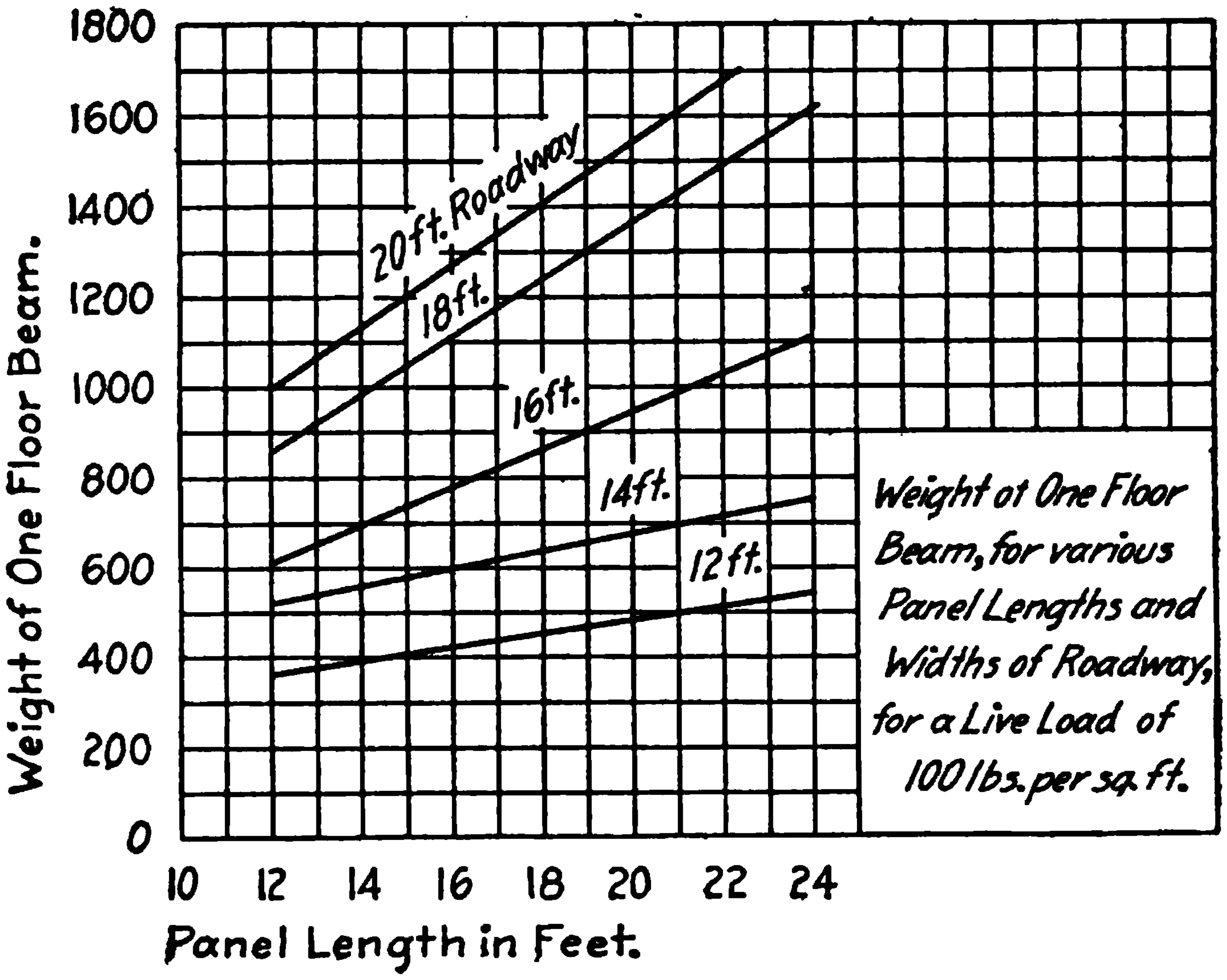


FIG. 35. TOTAL WEIGHT OF ONE FLOORBEAM FOR VARIOUS PANEL LENGTHS AND WIDTHS OF ROADWAY, FOR A LIVE LOAD OF 100 POUNDS PER SQUARE FOOT. (AMERICAN BRIDGE CO.)

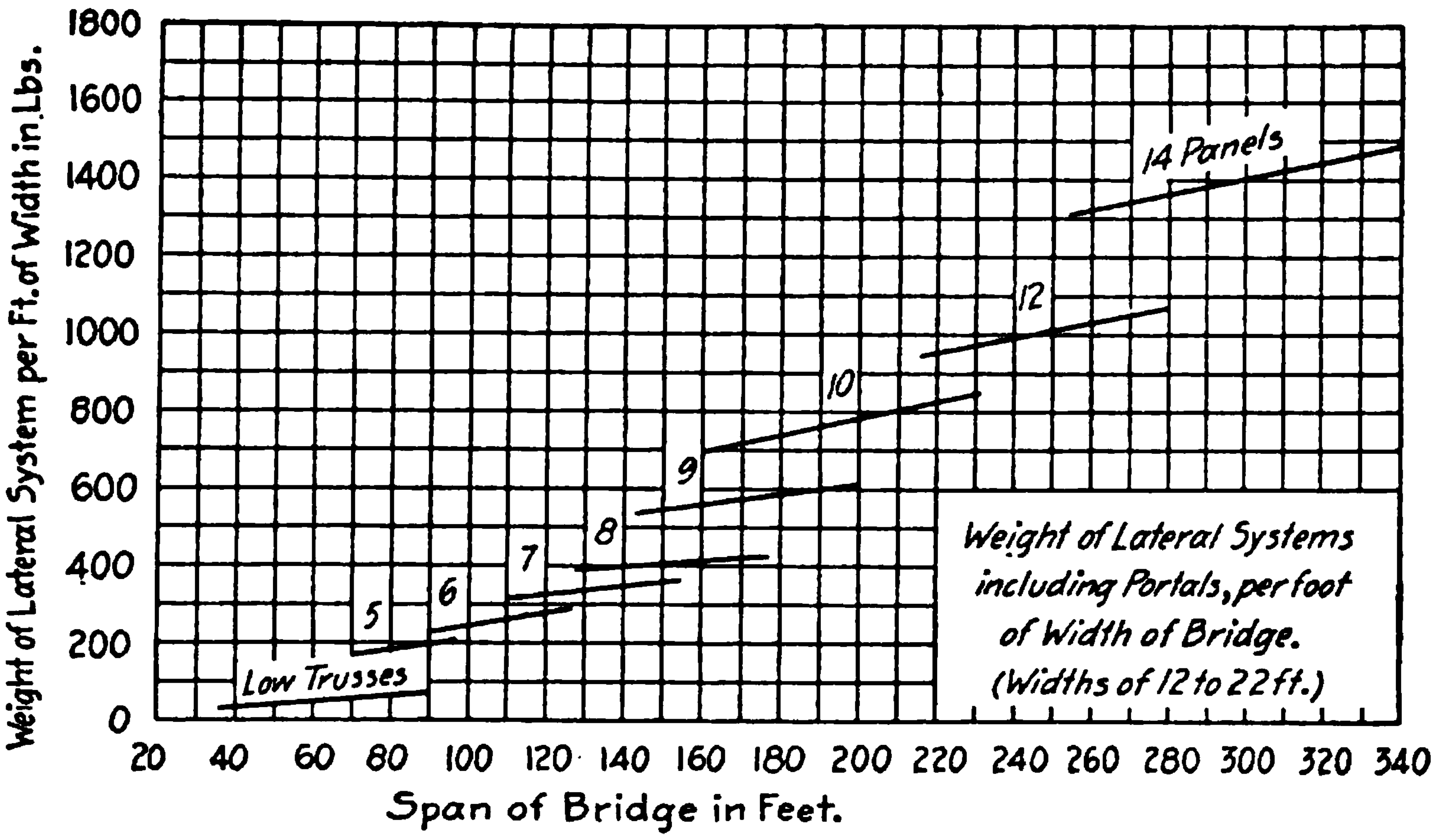


FIG. 36. WEIGHT OF UPPER AND LOWER LATERAL SYSTEMS AND THE PORTALS AND SWAY BRACING FOR THROUGH BRIDGES IN POUNDS PER FOOT OF WIDTH OF BRIDGE. (AMERICAN BRIDGE CO.)

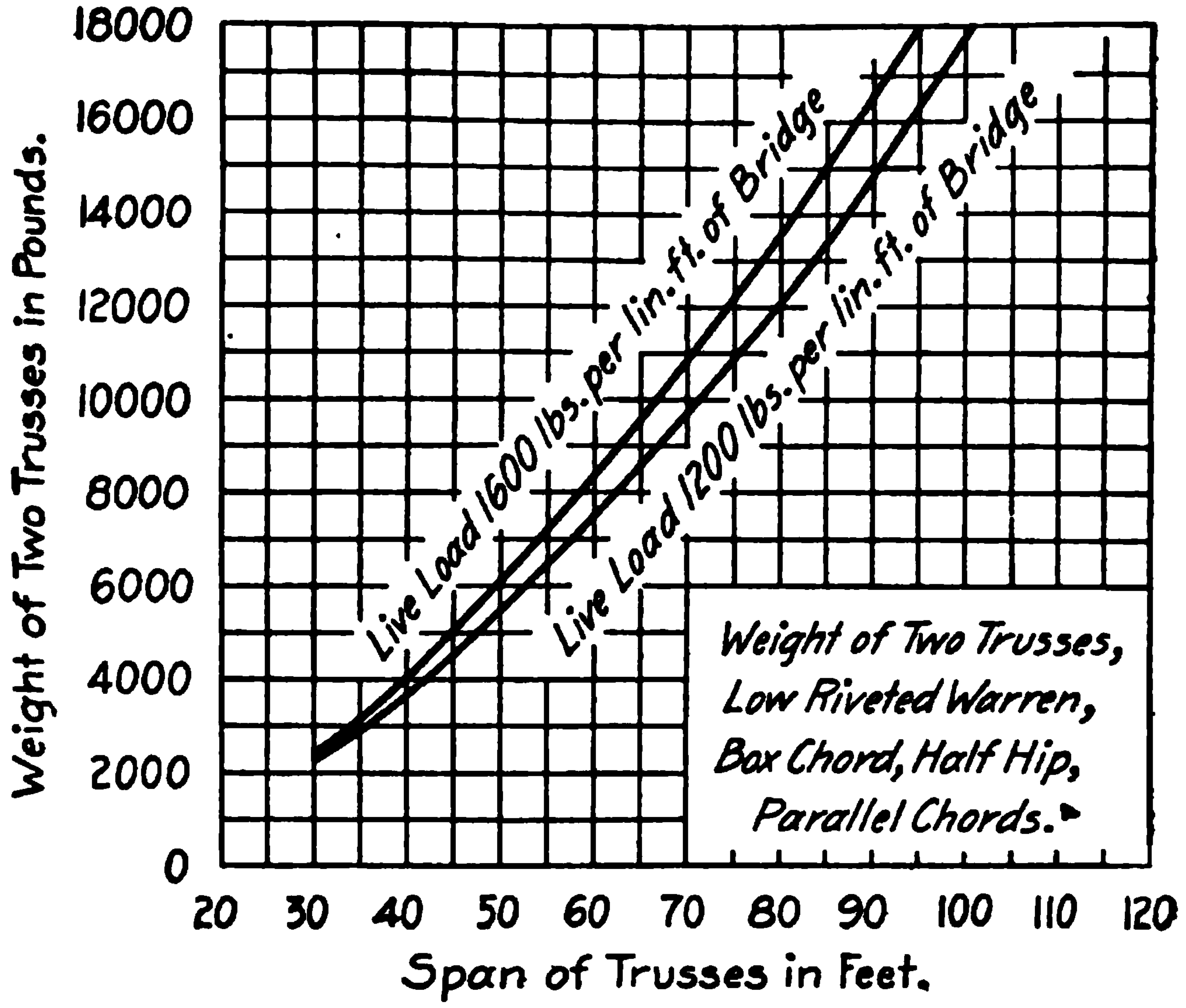


FIG. 37. TOTAL WEIGHT OF STEEL IN TWO TRUSSES OF A LOW, RIVETED, BOX-CHORD, WARREN TRUSS HIGHWAY BRIDGE (FIG. 7). (AMERICAN BRIDGE CO.)

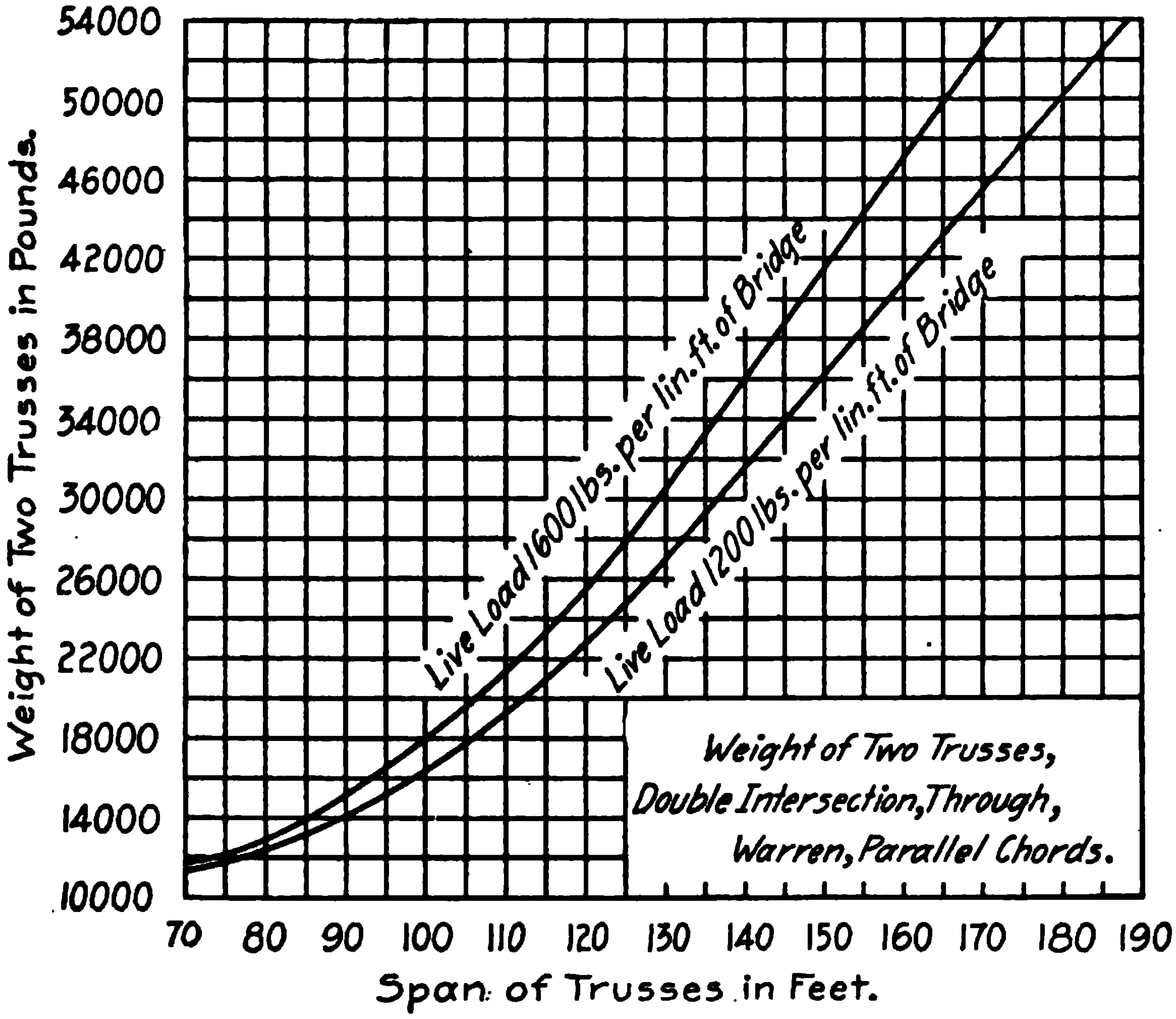


FIG. 38. TOTAL WEIGHT OF STEEL IN TWO TRUSSES OF A DOUBLE INTERSECTION, QUADRANGULAR, RIVETED, THROUGH WARREN TRUSS HIGHWAY BRIDGE (FIG. 13). (AMERICAN BRIDGE CO.)

lineal foot of bridge, and the floor system for a live load of 100 lbs. per square foot of floor, proceed as follows:

The total weight of two 80-foot trusses, from Fig. 34 = 12,700 lbs.
The total weight of 4 floorbeams for a panel length of 16 feet and a roadway of 18 feet, from Fig. 35 = 4 × 1,100 = 4,400 lbs.
The total weight of lateral systems, including portals, for

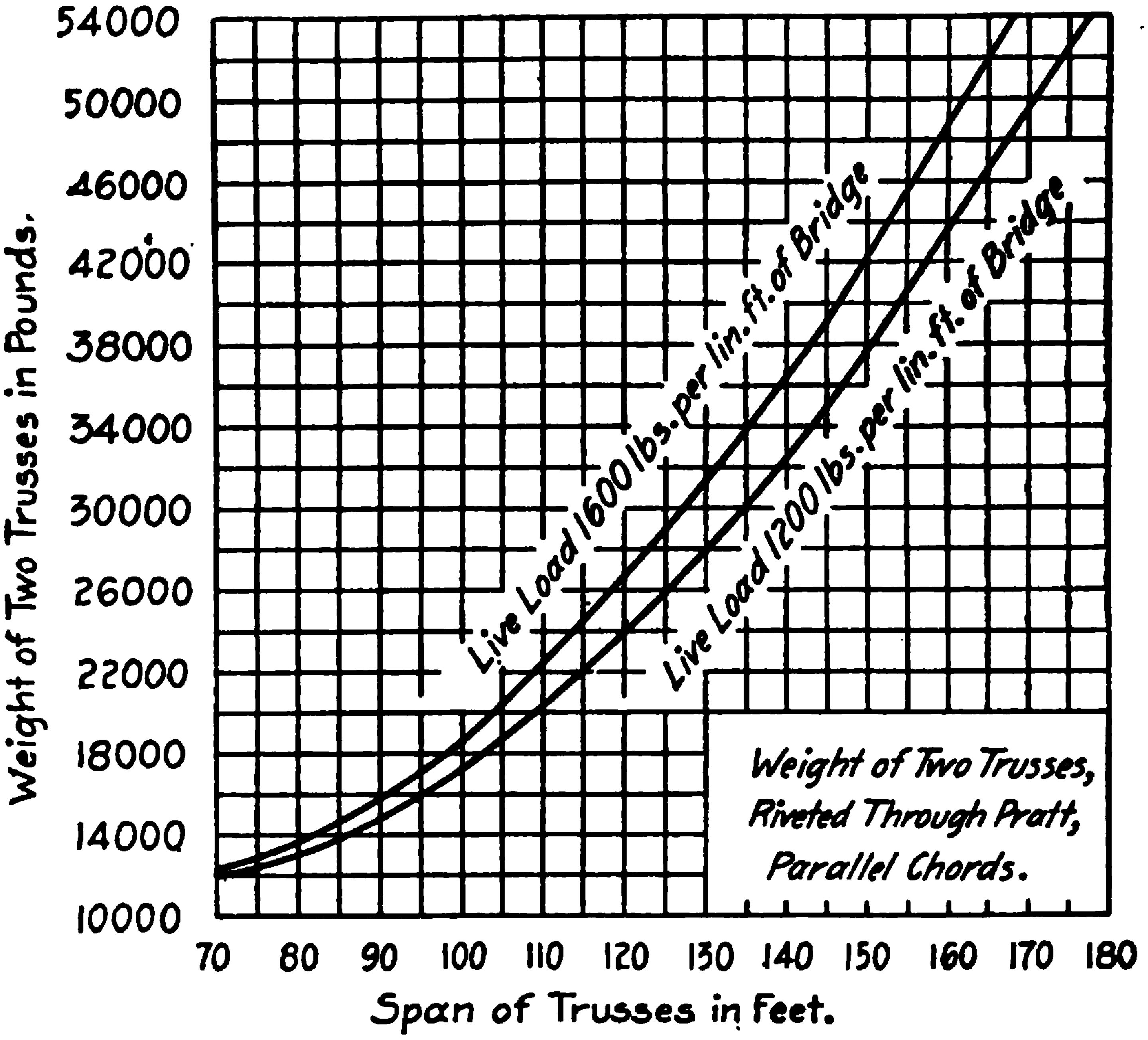


FIG. 39. TOTAL WEIGHT OF STEEL IN TWO TRUSSES OF A RIVETED, THROUGH PRATT TRUSS HIGHWAY BRIDGE (FIG. 10). (AMERICAN BRIDGE CO.)

a clear roadway of 18 feet, from Fig. 36 = 18 × 190 = 3,420 lbs.
Total weight of steel in bridge exclusive of joists and fence = 20,520 lbs.

The weights of two trusses of low, riveted, box-chord, Warren truss bridges, are given in Fig. 37; the weights of two trusses of double intersection, quadrangular, riveted, through Warren truss bridges are given in Fig. 38; the weights of two trusses of riveted, through Pratt truss bridges are given in Fig. 39; the weights of two trusses of pin-

connected, through Pratt truss bridges are given in Fig. 40; and the weights of two trusses of pin-connected, through curved-chord Pratt and Petit truss bridges are given in Fig. 41. The weights of floor-beams as given in Fig. 35, and the weights of lateral systems as given in Fig. 36, are to be used in connection with all diagrams giving the weights of two trusses.

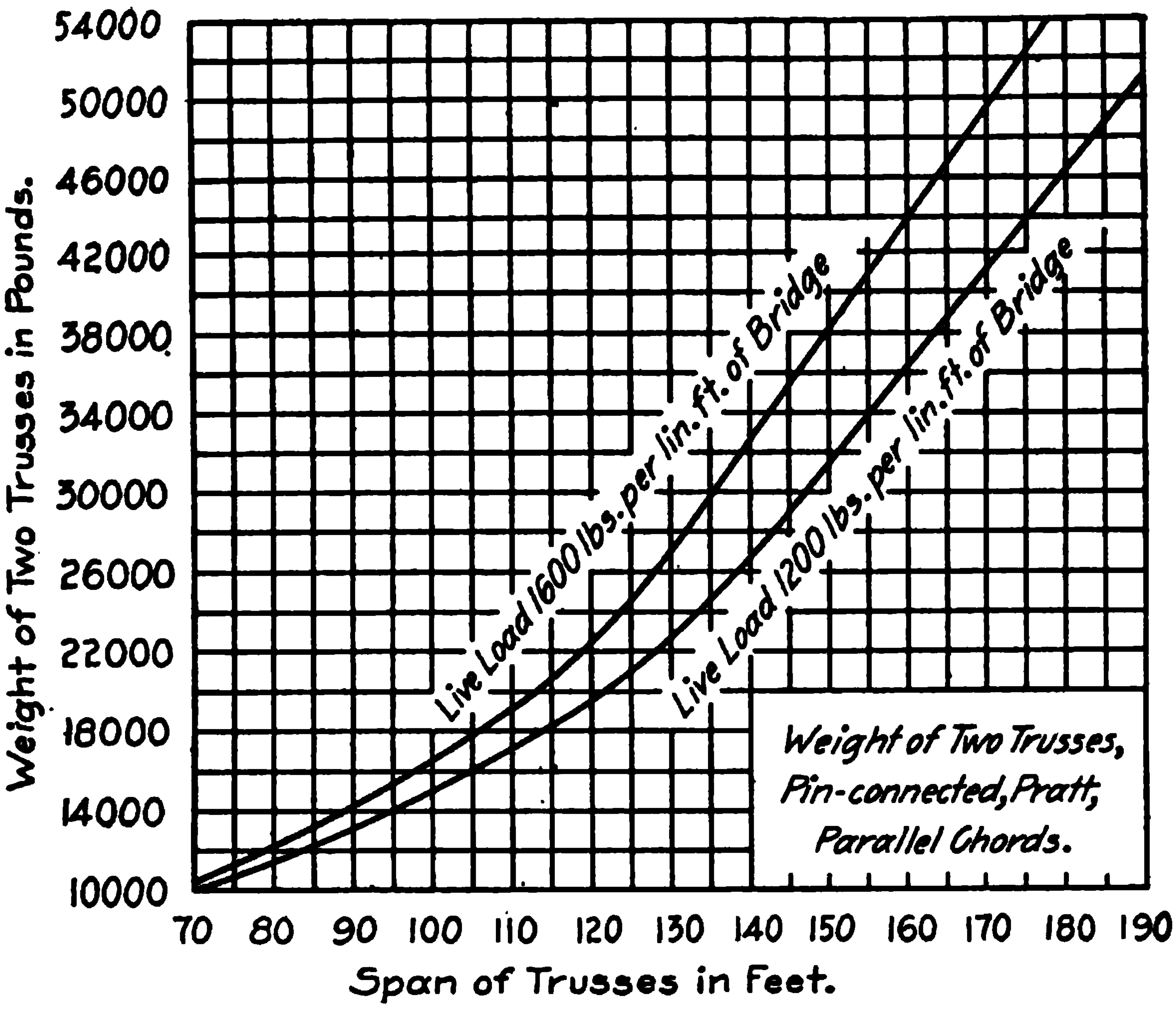


FIG. 40. TOTAL WEIGHT OF TWO TRUSSES OF A PIN-CONNECTED, THROUGH PRATT TRUSS HIGHWAY BRIDGE (FIG. 9). (AMERICAN BRIDGE CO.)

As a second example it is required to calculate the weight of a six-panel, pin-connected, Pratt truss bridge having a span of 110 feet and a roadway of 16 feet in the clear, the trusses having been designed for a live load of 1,200 lbs. per lineal foot of bridge, and a floor load of 100 lbs. per square foot of floor.

Total weight of two 110-ft. trusses, from Fig. 40 = 17,200 lbs.

Total weight of 5 floorbeams for a 16-ft. roadway and an 18⅓-ft. panel, from Fig. 35 = 900 × 5 = 4,500 lbs.

Total weight of lateral systems, including portals, from Fig. 36 = 16 × 250 =	4,000 lbs.
Total weight of bridge exclusive of joists and fence =	25,700 lbs.

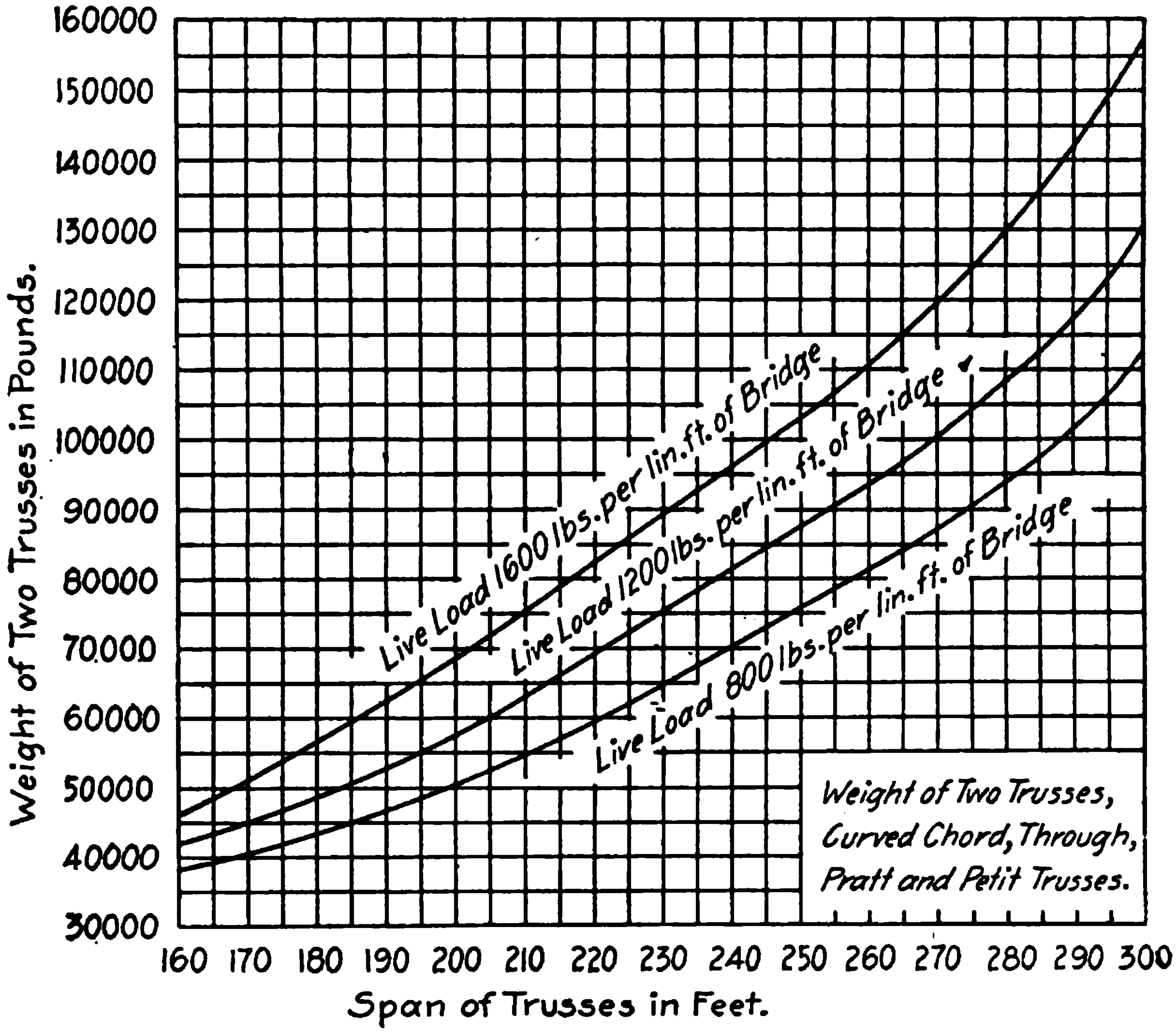


FIG. 41. TOTAL WEIGHT OF TWO TRUSSES OF A PIN-CONNECTED, CURVED CHORD, THROUGH PRATT AND PETIT HIGHWAY BRIDGE (FIG. 15 AND FIG. 18). (AMERICAN BRIDGE CO.)

From Fig. 30, the total weight of a 110-foot span, 6-panel, pin-connected Pratt truss bridge with a 16-foot clear roadway designed for a live load of 1,280 lbs. per lineal foot of bridge is approximately 25,200 lbs., which is a very good check.

As a third example it is required to calculate the weight of an 8-panel, pin-connected, Pratt truss highway bridge, having a span of 160 ft., a roadway of 16 ft. in the clear, the trusses having been designed for a live load of 1,440 lbs. per lineal foot of bridge, and a floor load of 100 lbs. per square foot of floor.

Total weight of two 160-ft. trusses, from Fig. 40, by interpolation = 40,000 lbs.

Total weight of 7 floorbeams for a 16-ft. roadway and a 20-ft. panel, from Fig. 35 = 950×7 = 6,650 lbs.

Total weight of lateral systems, including portals, from Fig. 36 = 16×410 = 6,560 lbs.

Total weight of bridge = 53,210 lbs.

WEIGHT OF BEAM BRIDGES.—The weights of beam bridges, as designed by the American Bridge Co. for different roadways, are given in Table VIII. The lengths of span for different loadings may be used as given in Table VII.

TABLE VII.

PERMISSIBLE SPAN OF BEAMS FOR DIFFERENT LOADINGS, ARRANGED AS IN TABLE VIII.

Size of Beams.	Maximum Span for a Live Load of 100 lbs. per sq. ft., or a 6 Ton Wagon.	Maximum Span for a Live Load of 125 lbs. per sq. ft., or a 15 Ton Road Roller.
6″	16′ 0″	14′ 0″
7″	19′ 0″	17′ 0″
8″	22′ 0″	20′ 0″
9″	25′ 0″	23′ 0″
10″	29′ 0″	27′ 0″
12″	35′ 0″	32′ 0″
15″	40′ 0″	40′ 0″

Weight of Steel Highway Plate Girder Bridges.—The weights of highway plate girder bridges, as designed by the Boston Bridge Works, are given in Figs. 42, 43 and 44. The through highway plate girder bridges, of which the weights are given in Fig. 42, were designed for a live load of 80 lbs. per square foot of floor surface for the girders and 100 lbs. per square foot of floor surface for the floor system, also for a 6-ton wagon.

The through highway plate girder bridges with sidewalks, of which the weights are given in Fig. 43, were designed for a live load of 80 lbs. per square foot on the roadway and 60 lbs. per square foot on the sidewalk for girders 30 ft. long and over; and 100 lbs. per square foot

TABLE VIII.

WEIGHTS OF BEAM BRIDGES. (AMERICAN BRIDGE CO.)

SIZE OF JOIST USED.	ITEMS.	WIDTH OF ROADWAY AND NUMBER OF JOISTS USED.				
		12' 0" 2 [s, 3 Is	14' 0" 2 [s, 4 Is	16' 0" 2 [s, 5 Is	18' 0" 2 [s, 6 Is	20' 0" 2 [s, 7 Is
	Joists, lbs. per lin. ft.	53	65	77	90	102
6" Is—12¼ lbs.	2 Wall channels.	260	300	340	380	420
6" [s—8 lbs.	1 Set bracing.	150	160	170	180	190
	Field bolts and clips.	51	55	59	63	67
	Joists, lbs. per lin. ft.	65	80	95	110	125
7" Is—15 lbs.	2 Wall channels.	265	305	345	385	425
7" [s—9¾ lbs.	1 Set bracing.	155	165	175	185	195
	Field bolts and clips.	.57	61	65	69	73
	Joists, lbs. per lin. ft.	77	95	113	131	149
8" Is—18 lbs.	2 Wall channels.	270	310	350	390	430
8" [s—11¼ lbs.	1 Set bracing.	160	170	180	190	200
	Field bolts and clips.	65	69	73	77	81
	Joists, lbs. per lin. ft.	90	111	132	153	174
9" Is—21 lbs.	2 Wall channels.	275	315	355	395	435
9" [s—13¼ lbs.	1 Set bracing.	165	175	185	195	205
	Field bolts and clips.	67	72	77	82	87
	Joists, lbs. per lin. ft.	105	130	155	180	205
10" Is—25 lbs.	2 Wall channels.	280	320	360	400	440
10" [s—15 lbs.	1 Set bracing.	170	180	190	200	210
	Field bolts and clips.	75	80	85	90	95
	Joists, lbs. per lin. ft.	136	167	199	230	262
12" Is—31½ lbs.	2 Wall channels.	290	330	370	410	450
12" [s—20½ lbs.	2 Sets bracing.	350	370	390	410	430
	Field bolts and clips.	118	129	140	151	162
	Joists, lbs. per. lin. ft.	146	181	216	251	286
12" Is—35 lbs.	2 Wall channels.	290	330	370	410	450
12" [s—20½ lbs.	2 Sets bracing.	350	370	390	410	430
	Field bolts and clips.	118	129	140	151	162
	Joists, lbs. per lin. ft.	161	201	241	281	321
12" Is—40 lbs.	2 Wall channels.	290	330	370	410	450
12" [s—20½ lbs.	2 Sets bracing.	350	370	390	410	430
	Field bolts and clips.	123	135	147	159	171
	Joists, lbs. per lin. ft.	192	234	276	318	360
15" Is—42 lbs.	2 Wall channels.	305	345	385	425	465
15" [s—33 lbs.	2 Sets bracing.	360	380	400	420	440
	Field bolts and clips.	138	151	164	177	190
Lumber: Ft. B. M. per lin. ft. span.		40	46	52	58	64

Railing: Total weight of 2 sides = 33 lbs. × length in feet + 100 lbs.
Extreme length of span = clear span + 2 ft. (ordinarily).
Bracing: 12" and 15" beams have 2 sets; all other sizes 1 set.

on roadway and sidewalk for girders under 30 ft.; also for a 6-ton wagon.

The deck highway plate girder bridges without walks, of which the weights are given in Fig. 44, were designed for the same loads as

FIG. 42. TOTAL WEIGHT OF STEEL IN THROUGH PLATE GIRDER HIGHWAY BRIDGES, EXCLUSIVE OF JOISTS AND FENCE. (BOSTON BRIDGE WORKS.)

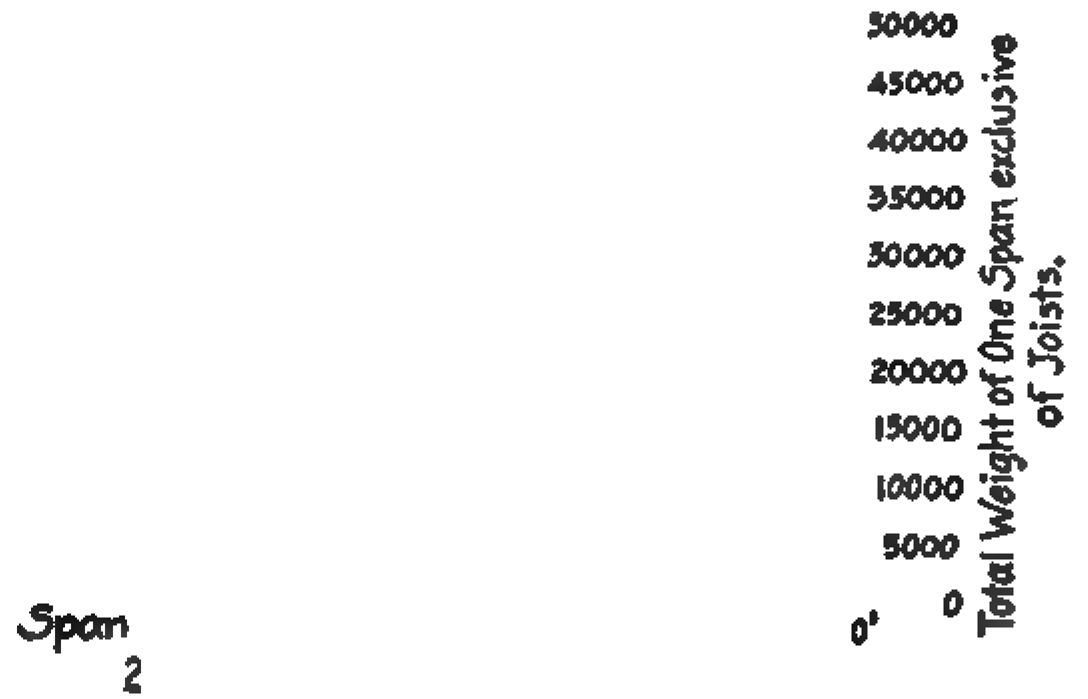


FIG. 43. TOTAL WEIGHT OF STEEL IN THROUGH PLATE GIRDER HIGHWAY BRIDGES WITH SIDEWALKS, EXCLUSIVE OF JOISTS AND FENCE. (BOSTON BRIDGE WORKS.)

the through plate girder bridges in Fig. 42. The allowable unit stresses were the same as for the truss bridges in Figs. 32 and 33.

FIG. 44. TOTAL WEIGHT OF STEEL IN DECK PLATE GIRDER HIGHWAY BRIDGES, EXCLUSIVE OF JOISTS AND FENCE. (BOSTON BRIDGE WORKS.)

TABLE IX.

WEIGHT OF ONE STEEL HIGHWAY PLATE GIRDER FOR DIFFERENT LOADINGS PER LINEAL FOOT OF ONE GIRDER. (BOSTON BRIDGE WORKS.)

Span Feet.	Class I.			Class II.			Class III.		
	Live Load.	Dead Load.	Weight, Pounds.	Live Load.	Dead Load.	Weight, Pounds.	Live Load.	Dead Load.	Weight, Pounds.
15	640	230	600	1,800	480	900	3,000	790	1,000
20		235	800		490	1,300		805	1,700
25		240	1,000		500	2,000		820	2,600
30		245	1,400		510	2,800		835	3,700
35		250	1,900		520	3,800		850	5,100
40		255	2,600		530	5,000		865	6,600
45		260	3,300		540	6,400		880	8,400
50		265	4,200		550	8,000		895	10,400
55		270	5,200		560	9,700		910	12,500
60		275	6,300		570	11,700		925	14,900

The weights of one steel highway plate girder for different loads per lineal foot of girder, and different spans are given in Table IX. The compression flanges were made with the same section as the tension flanges. The live and dead loads are given in pounds per lineal foot of one girder.

WEIGHT OF STEEL ELECTRIC RAILWAY BRIDGES.—The weights of the structural steel in electric railway bridges, as designed by the Boston Bridge Works, are given in Fig. 45.

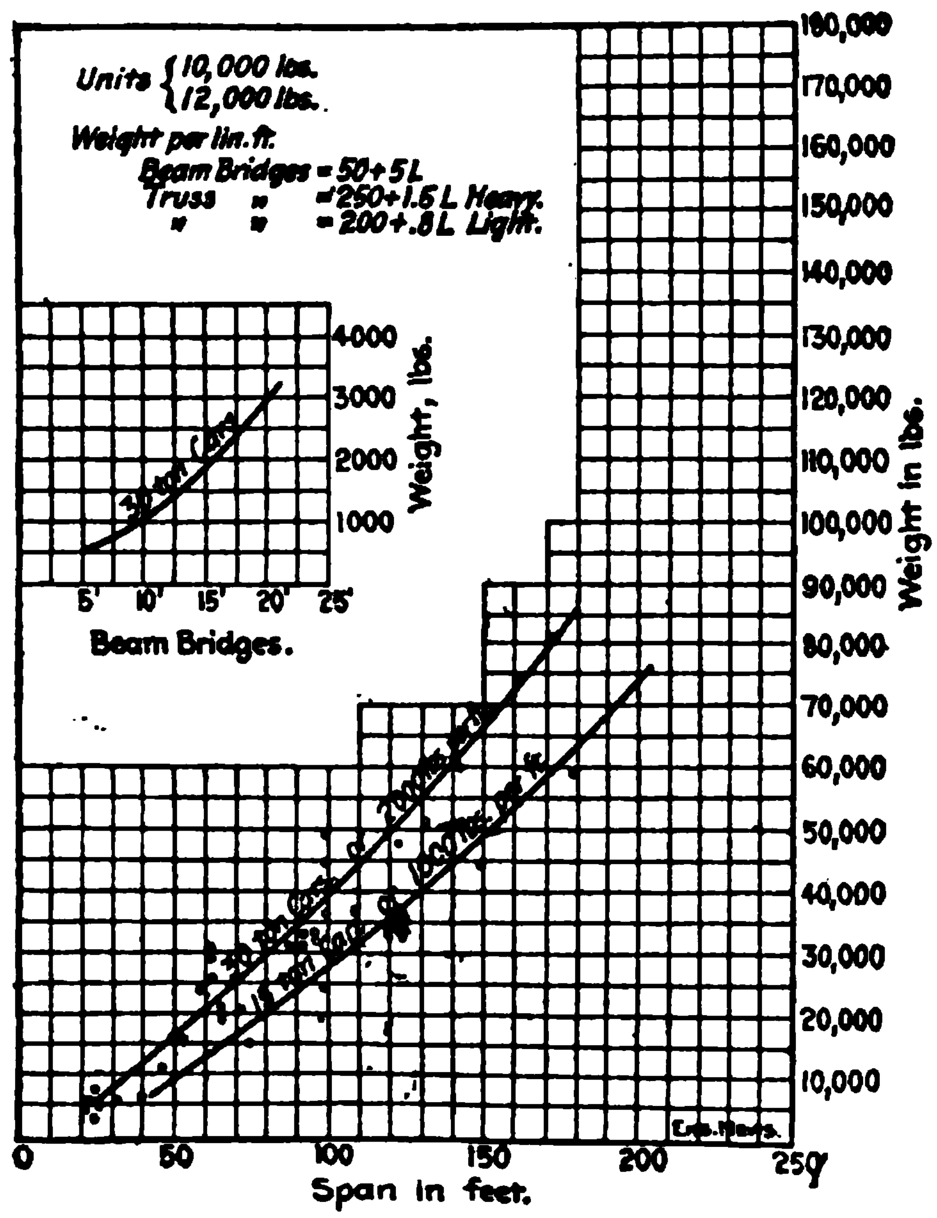


FIG. 45. TOTAL WEIGHT OF STEEL IN ELECTRIC RAILWAY BRIDGES, EXCLUSIVE OF FENCE. (BOSTON BRIDGE WORKS.)

WEIGHT OF STEEL RAILWAY BRIDGES.—The weight of the structural steel in a railway bridge varies with the span, the type of truss, the loading and with the specifications. The formula proposed by Waddell and Hedrick and published in Merriman and Jacoby's Roofs and Bridges, Part I, is

$$w = 8.63\ (l + 1.3W - 140)$$

where w is the weight of steel in pounds per lineal foot of span, l is the span in feet, and W is the weight in net tons of each of the two locomotives that precede the uniform train load.

For Cooper's E 40 loading the weight is then given by the formula

$$w = 8.63\ (l + 44.6) \text{ lbs.} \qquad \text{(a)}$$

and for Cooper's E 50 loading the weight is given by the formula

$$w = 8.63\ (l + 90.75) \text{ lbs.} \qquad \text{(b)}$$

To obtain the total dead load, add from 400 to 500 lbs. per lineal foot for the ties and track.

FLOOR SYSTEMS FOR HIGHWAY BRIDGES.—The American Bridge Co.'s Specifications for Steel Highway Bridges, 1901, include the following provisions:

§ 1. **Classification.**—Bridges under these specifications are divided into six classes, viz.:

Class A.—For city traffic.

Class B.—For suburban or interurban traffic with heavy electric cars.

Class C.—For country roads, with light electric cars or heavy highway traffic.

Class D.—For country roads, with ordinary highway traffic.

Class E_1.—For heavy electric street railways only.

Class E_2.—For light electric street railways only.

§ 8. **Spacing of Trusses.**—The width between centers of trusses shall in no case be less than one-twentieth of the span between centers of end pins or shoes.

§ 11. **Floorbeams.**—All floorbeams in through bridges shall be riveted to the main girders.

§ 12. **Stringers.**—Steel stringers shall preferably be riveted to the web of the floorbeams.

Wooden joists shall not be less than 3 inches thick, shall be spaced not more than 2½ feet between centers, and shall be dapped over the seat angles or floorbeams to exact level. In the latter case they shall lap by each other over the full width of the floorbeam, and shall be separated ½ inch for free circulation of air.

§ 13. For single thickness the roadway planks shall not be less than 3 inches thick, nor less than one-twelfth of the distance between stringers, and shall be laid transversely with ¼ inch openings.

§ 15. **Wheel-guards.**—Wheel-guards of a cross-section not less than 6 inches by 4 inches shall be provided on each side of the roadway. They shall be blocked up from the floor plank with blocks 2 inches by 6 inches by 12 inches long, not over 5 feet apart center to center, held in place by one ¾ inch bolt passing through the center of each blocking piece and securely fastened to the stringer below. The wheel-guards shall be spliced with half and half joints with 6 inches lap over a blocking piece.

§ 16. **Footwalk Planks.**—The footwalk planks shall not be less than 2 inches thick nor more than 6 inches wide, spaced with ½ inch openings.

§ 17. All plank shall be laid with the heart side down; shall have full and even bearing on and be firmly attached to the stringers.

§ 18. **Solid Floor.**—For bridges of Classes A and B a solid floor, consisting of stone, asphalt, etc., on a concrete bed, is recommended. For this case the flooring will consist of buckle-plates or corrugated sections, and the concrete bed shall be at least 3 inches thick for the roadway and 2 inches thick for the footwalk, over the highest point to be covered, not counting rivet or boltheads.

§ 19. **Buckle Plates.**—Buckle plates shall not be less than $\frac{5}{16}$ inch thick for the roadway and ¼ inch thick for the footwalk.

§ 20. **Curbs.**—For solid floor the curb holding the paving and acting as a wheel-guard on each side of the roadway shall be of stone or steel projecting about 6 inches above the finished paving at the gutter. The curb shall be so arranged that it can be removed and replaced when worn or injured. There shall also be a metal edging strip on each side of the footwalks to protect and hold the paving in place.

§ 21. **Drainage.**—Provision shall be made for drainage clear of all parts of the metal work.

§ 22. **Floor of Class E_1 and E_2.**—The floor of bridges of Classes E_1 and E_2 shall consist of cross-ties not less than 6 inches by 6 inches, spaced with openings not exceeding 6 inches and securely fastened to the stringers by bolts. There shall be guard timbers not less than 6 inches by 6 inches on each side of each track, with the inner faces not less than 9 inches from center of rail. They shall be notched 1 inch over every tie, and fastened to every tie.

§ 23. **Dead Load.**—In determining the weights of the structure for the purpose of calculating stresses, the weight of timber shall be assumed at 4 lbs. per foot B. M., the weight of concrete and asphaltum at 130 lbs., of paving brick at 150 lbs. and of granite at 160 lbs., per cubic foot.

The rails, fastenings, splice and guard timbers of street railway tracks, resting on cross-ties, shall be assumed as weighing 100 lbs. per lineal foot of track.

For additional specifications for floor systems of highway bridges,

see the author's "Specifications for Steel Highway Bridges," in Appendix I.

FLOOR SYSTEMS FOR ELECTRIC RAILWAY BRIDGES. —Specifications for Steel Electric Railway Bridges by Mr. C. C. Schneider, M. Am. Soc. C. E., contain the following provisions:

§ 1. **Classification.**—Bridges under these specifications are divided into three classes, viz.:

Class A.—For heavy electric railways carrying freight.

Class B.—For city traffic, including elevated railways.

Class C.—For light suburban railways.

§ 4. **Clearance.**—In all through bridges the clear width from the center of the track shall not be less than 7 ft. at a height of 1 ft. 6 in. above the tops of the rails where the tracks are straight. The width shall be increased to provide the same minimum clearance on curves, allowance being made for super-elevation of rails.

§ 5. **Head-room.**—The clear head-room for all bridges shall not be less than 15 ft. above the top of the rails.

§ 6. **Spacing Trusses.**—The width center to center of trusses shall in no case be less than one-twentieth of the effective span.

Spacing of Stringers.—Stringers shall be spaced generally not less than 6½ ft. centers, in high viaducts not less than 8 ft.

§ 7. **Skew Bridges.**—Ends of deck plate girders and track stringers of skew bridges at abutments shall be square to the track.

§ 8. **Ties.**—Wooden tie floors, where used, shall be proportioned to carry the maximum wheel load with 50 per cent impact distributed over three ties; fiber stress on ties not to exceed 2,000 lbs. per square inch, and the length to be not less than the total distance over the outer edge of the supports plus 12 ins. They shall be not less than 7 ins. wide and spaced with not more than 6-in. openings, and shall be notched to a tight fit over the supporting girders, depth of notch to be not more than 1½ ins.

§ 9. **Guard Timbers.**—Guard timbers shall be not less than 5 ins. by 8 ins., laid with the 8-in. face down, notched over each tie and securely fastened.

§ 10. **Dead Load.**—The dead load shall consist of the estimated weight of the entire suspended structure, assuming timber to weigh

4½ lbs. per foot B. M., and rails and fastenings 100 lbs. per lineal foot of track.

FLOOR SYSTEMS FOR RAILWAY BRIDGES.—The American Bridge Co.'s Specifications for Railway Bridges, 1901, include the following provisions:

§ 8. **Wooden Floor.**—The floor shall consist of cross ties 8 inches by 8 inches if the stringers are placed 6½ feet between centers. For greater distances they are to be proportioned to carry the maximum wheel load distributed over three ties, the fiber stress on the timber not to exceed 1,000 lbs. per square inch. The ties shall be spaced with openings not exceeding 6 inches, and shall be notched down ½ inch, and have a full and even bearing on stringers.

§ 10. **Guard Rails.**—There shall be guard timbers 6 inches by 8 inches on each side of each track, with their inner faces not less than 3 ft. 3 ins. from center of track. They shall be notched 1 inch over every tie, and shall be fastened to every third tie and at each splice by a ¾ inch bolt. Splices shall be over floor timbers with half-and-half joints of 6 inches lap.

§ 11. The floor timbers and guards must be continued over piers and abutments.

§ 12. On curves the outer rails shall be elevated as may be required.

§ 13. **Dead Load.**—In determining the weight of the structure for the purpose of calculating stresses, the weight of timber shall be assumed at 4½ lbs. per foot B. M., and the weight of rails, spikes and joints at 100 lbs. per lineal foot of track.

LIVE LOADS FOR HIGHWAY BRIDGES.—The live loads specified in the American Bridge Co.'s Specifications for Highway Bridges, and in Cooper's Specifications for Steel Highway and Electric Railway Bridges, 1901 edition, are the same and are as given in the following paragraphs, extracted from the American Bridge Co.'s 1901 Specifications:

§ 24. **Live Load.**—The bridges of the different classes shall be designed to carry, in addition to their own weight and that of the floor, a moving load, either uniform or concentrated, or both, as specified below, placed so as to give the greatest stress in each part of the structure.

Class A. City Bridges.—For the floor and its supports, on each

street car track or on any part of the roadway, a concentrated load of 24 tons on two axles 10 ft. centers and 5 ft. gage (assumed to occupy a width of 12 ft.), and upon the remaining portion of the floor, including footwalks, a load of 100 lbs. per square foot.

For the trusses, for spans up to 100 ft., 1,800 lbs. per lineal foot of each car track (assumed to occupy 12 ft. in width), and 100 lbs. per square foot for the remaining floor surface; for spans of 200 ft. and over, 1,200 lbs. for each lineal foot of track and 80 lbs. per square foot of floor; proportionately for intermediate spans.

Class B. Suburban or Interurban Bridges.—For the floor and its supports on any part of the roadway, a concentrated load of 12 tons on two axles 10 ft. centers and 5 ft. gage (assumed to occupy a width of 12 ft.), or on each street car track a concentrated load of 24 tons on two axles 10 ft. centers; and upon the remaining portion of the floor, including footwalks, a load of 100 lbs. per square foot.

For the trusses, for spans up to 100 ft., 1,800 lbs. per lineal foot of each car track and 80 lbs. per square foot for the remaining floor surface; for spans of 200 ft. and over, 1,200 lbs. for each lineal foot of track and 60 lbs. per square foot of floor; proportionately for intermediate spans.

Class C. Heavy Country Highway Bridges.—For the floor and its supports, on any part of the roadway, a concentrated load of 12 tons on two axles 10 ft. centers and 5 ft. gage (assumed to occupy a width of 12 ft.), or on each street car track a concentrated load of 18 tons on two axles 10 ft. centers; and upon the remaining portion of the floor, including footwalks, a load of 100 lbs. per square foot.

For the trusses, same as for Class B, except load on car track for spans up to 100 ft. will be 1,200 lbs. and for spans of 200 feet and over 1,000 lbs.

Class D. Ordinary Country Highway Bridges.—For the floor and its supports, a load of 80 lbs. per square foot of total floor surface or 6 tons on two axles 10 ft. centers and 5 ft. gage.

For the trusses, a load of 80 lbs. per square foot of total floor surface for spans up to 75 feet; and 55 lbs. for spans of 200 ft. and over; proportionately for intermediate spans.

Class E_1. Bridges for Heavy Electric Steel Railways Only.—For the floor and its supports, on each track a load of 24 tons on two axles 10 ft. centers.

For the trusses, a load of 1,800 lbs. per lineal foot of each car

track for spans up to 100 ft.; and a load of 1,200 lbs. for spans of 200 ft. and over; proportionately for intermediate spans.

Class E_2. Bridges for Light Electric Street Railways Only.—For the floor and its supports, on each track a load of 18 tons on two axles 10 ft. centers.

For the trusses, a load of 1,200 lbs. per lineal foot of each car track for spans up to 100 ft.; and a load of 1,000 lbs. for spans of 200 ft. and over; proportionately for intermediate spans.

The Author's Specifications.—The loadings specified in the author's "Specifications for Steel Highway Bridges" given in Appendix I, are the same as specified by the American Bridge Co. and by Cooper, except that ordinary highway bridges are classified under D_1 and D_2; and Schneider's loadings have been used in place of E_1 and E_2.

Waddell's Loadings.—Mr. J. A. L. Waddell in his Highway Bridge Specifications divides highway bridges into three classes, viz.: Class A, bridges designed for heavy service; Class B, bridges designed

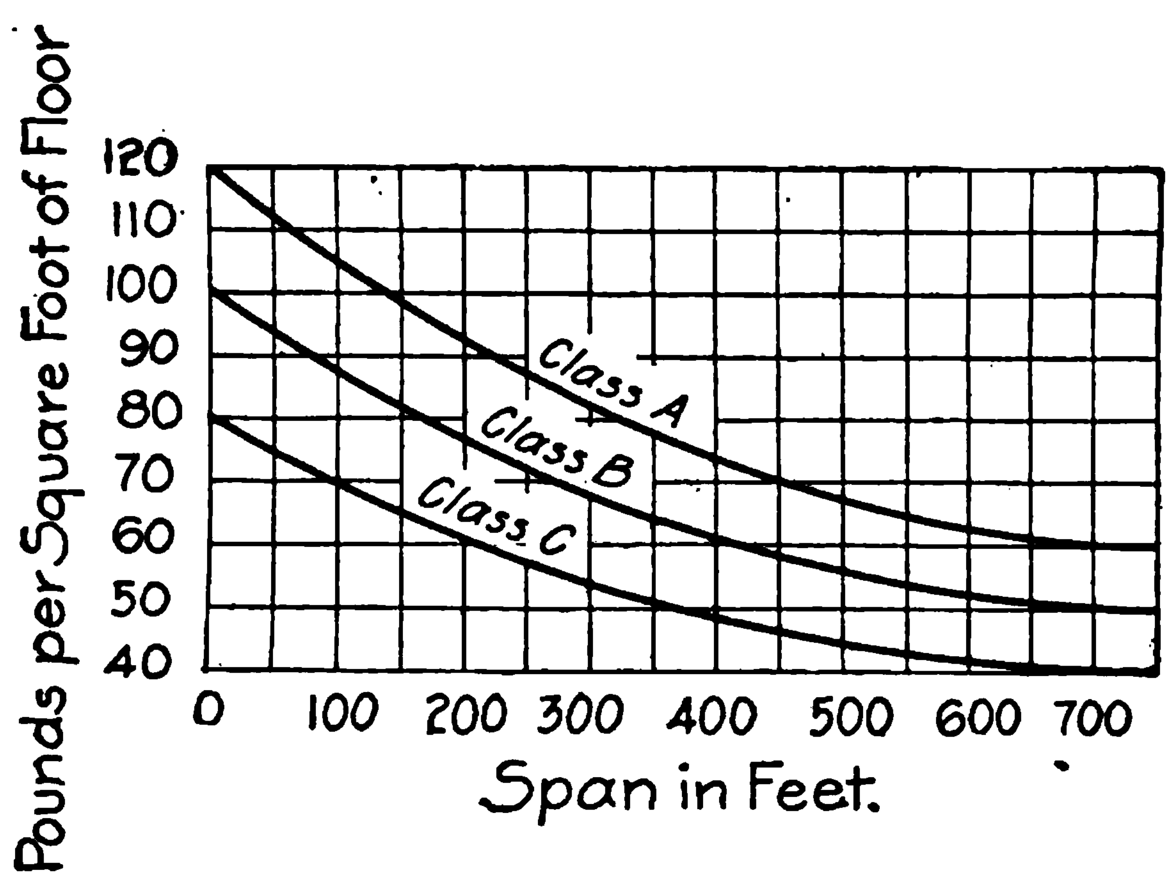


FIG. 46. WADDELL'S LIVE LOADS FOR HIGHWAY BRIDGES.

for moderately heavy loads; and Class C, bridges designed for light country service. The uniform load per square foot is to be taken from Fig. 46, and in addition the concentrated loads given below are to be considered.

Class A.—A road roller weighing 30,000 lbs., of which 12,000 lbs. are concentrated upon the roller in front of the machine and 9,000

lbs. on each of the wheels at the rear, the distance between the central planes of these wheels being 5 ft. and that between the axles of the wheels and the axle of the front roller 11 ft. The width of the front roller is to be 4 ft. and that of each rear wheel 1 ft. 8 ins.

Class B.—A concentrated load of 16,000 lbs., equally distributed upon two pairs of wheels, the axles of which are 8 ft. apart, and the central planes of the wheels 6 ft. apart.

Class C.—A concentrated load of 10,000 lbs., distributed in the same manner as for Class B.

The road roller is assumed to be equally divided between all the joists that it can cover, and the wheel loads for Classes B and C equally between two joists.

LIVE LOADS FOR ELECTRIC RAILWAY BRIDGES.—The live loads specified by Mr. C. C. Schneider, M. Am. Soc. C. E., in his Specifications for Electric Railway Bridges are as follows:

§ 11. **Moving Load.**—The moving load shall consist of one of the following classes:

Class A.—On each track a series of concentrations consisting of two pairs of trucks, the axles of the pairs being spaced 5 ft. centers, while the distance between centers of interior axles is 10 ft., the pairs of trucks being spaced 15 ft. centers. The axles are loaded with a load of 40,000 lbs., making a total of 160,000 lbs. Or a uniform load of 6,000 lbs. per lineal foot for all spans up to 50 ft., reduced to 4,500 lbs. per lineal foot for spans of 200 ft. and over, and proportionately for intermediate spans.

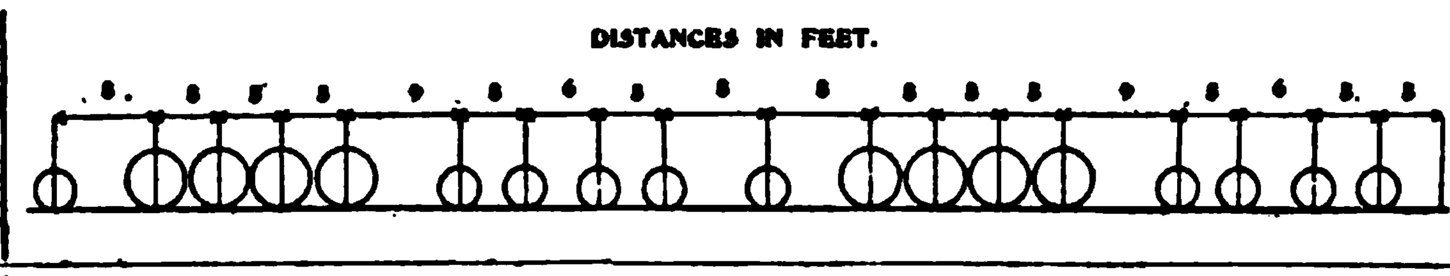
Class B.—On each track a series of concentrations consisting of two pairs of trucks, the axles of the pairs being spaced 5 ft. centers, while the distance between centers of interior axles is 10 ft., the pairs of trucks being spaced 15 ft. centers. The axles are loaded with a load of 25,000 lbs., making a total load of 100,000 lbs. Or a uniform load of 3,500 lbs. per lineal foot for all spans up to 50 ft., reduced to 2,000 lbs. per lineal foot for spans of 200 ft. and over, and proportionately for intermediate spans.

Class C.—On each track a series of concentrations consisting of two pairs of trucks, the axles of the pairs being spaced 5 ft. centers, while the distance between centers of interior axles is 10 ft., the pairs of trucks being spaced 15 ft. centers. The axles are loaded with a load of 20,000 lbs., making a total load of 80,000 lbs. Or a uniform load of 2,500 lbs. per lineal foot for all spans up to 50 ft., reduced to

1,500 lbs. per lineal foot for spans of 200 ft. and over, and proportionately for intermediate spans.

LIVE LOADS FOR RAILWAY BRIDGES.—The live loads on railway bridges are properly a series of moving concentrated loads. The loads may be considered to consist of a series of wheel loads due to one or more locomotives, followed by a uniform train load; or as an equivalent uniform load.

Concentrated Loads.—The most common wheel concentration loading is Cooper's Conventional System, in which two consolidation locomotives are followed by a uniform train load. The spacings for



Class																			Uniform Load.
E 50	25000	50000	50000	50000	50000	32500	32500	32500	32500	25000	50000	50000	50000	50000	32500	32500	32500	32500	5000 lbs. per lin. ft.
E 45	22500	45000	45000	45000	45000	29250	29250	29250	29250	22500	45000	45000	45000	45000	29250	29250	29250	29250	4500 lbs. per lin. ft.
E 40	20000	40000	40000	40000	40000	26000	26000	26000	26000	20000	40000	40000	40000	40000	26000	26000	26000	26000	4000 lbs. per lin. ft.
E 30	15000	30000	30000	30000	30000	19500	19500	19500	19500	15000	30000	30000	30000	30000	19500	19500	19500	19500	3000 lbs. per lin. ft.

FIG. 47. COOPER'S LOADINGS.

the wheels of all loadings are constant, the loads on the wheels being proportional in each case. Cooper's loadings are shown in Fig. 47. It will be seen that Cooper's E 50 loading has the same wheel spacings as E 40, all loads being 5/4 of the loads for E 40.

In bridges designed for Class E 40 loading and under the floor system must in addition be designed for two moving loads of 100,000 lbs. each, spaced 6′ 0″ apart on each track. The corresponding loads for Class E 50 are 120,000 lbs. with the same spacing. The American Railway Engineering and Maintenance of Way Associa-

tion has adopted Cooper's loadings, except that the special loads are spaced 7′ 0″.

Equivalent Uniform Loads.—An equivalent uniform load is one which approximately produces the same stresses as are produced by a series of concentrated loads. The equivalent uniform load for bending moment in a bridge is the uniform load that will produce the same bending moment at the quarter point of a bridge as the actual wheel loads. The equivalent uniform loads for bridge trusses up to 300 ft. span for Cooper's E 40 loading are given in Fig. 48. For a discussion of the stresses in railway bridge trusses, see Chapter VI.

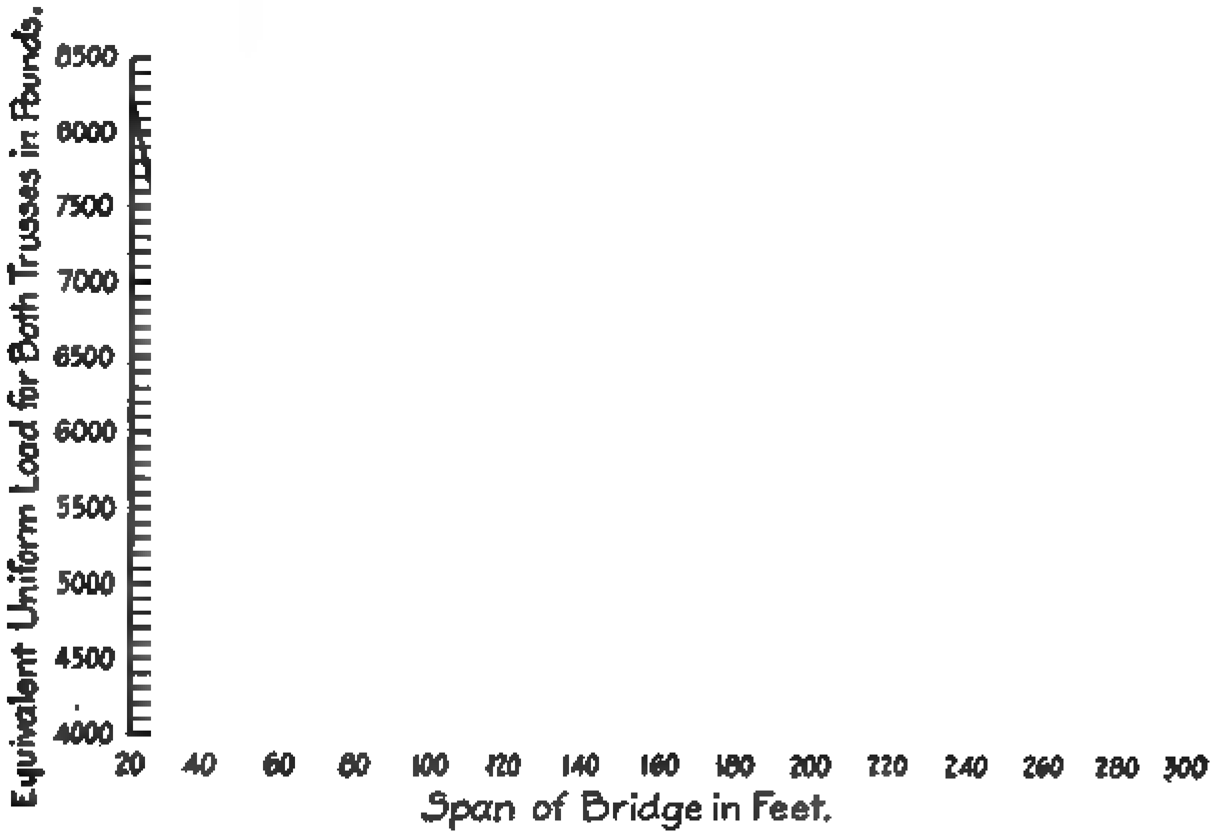


FIG. 48. EQUIVALENT UNIFORM LIVE LOAD FOR COOPER'S E 40 LOADING FOR RAILWAY BRIDGES.

WIND LOADS.—The wind blows against the bridge and causes stresses. The wind loads, as given in Cooper's Specifications for Steel Highway and Electric Railway Bridges, 1901 edition, are as follows:

§ 39. **Wind Loads.**—To provide for wind stresses and vibrations, the top lateral bracing in deck bridges, and the bottom bracing in through bridges, shall be proportioned to resist a lateral force of 300 lbs. for each foot of the span; 150 lbs. of this to be treated as a moving load.

The bottom lateral bracing in deck bridges, and the top lateral bracing in through bridges, shall be proportioned to resist a lateral force of 150 lbs. for each lineal foot of span. For spans exceeding 300 ft., add in each of the above cases 10 lbs. additional for each additional 30 ft.

§ 40. In trestle towers the bracing and columns shall be proportioned to resist the following lateral forces, in addition to the stresses from dead and live loads:

The trusses loaded or unloaded, the lateral pressures specified above and a lateral pressure of 100 lbs. for each vertical lineal foot of the trestle bents.

The wind loads, as given in the American Bridge Co.'s Specifications for Steel Highway Bridges, 1901 edition, are as follows:

§ 26. **Wind Loads.**—The wind pressure shall be assumed acting in either direction horizontally:

First.—At 30 lbs. per square foot on the exposed surface of all trusses and the floor as seen in elevation, in addition to a horizontal live load of 150 lbs. per lineal foot of the span moving across the bridge.

Second.—At 50 lbs. per square foot on the exposed surface of all trusses and the floor system. The greatest result shall be assumed in proportioning the parts.

The wind loads, as given in Mr. C. C. Schneider's Specifications for Electric Railway Bridges, are as follows:

§ 13. **Wind Loads.**—All spans shall be designed for a lateral force on the loaded chord of 200 lbs. per lineal foot, plus 10 per cent of the specified train load on one track, and 200 lbs. per lineal foot of the unloaded chord; these forces being considered as moving. The laterals throughout the unloaded chord shall be the same section as required by the end panels.

§ 14. Viaduct towers shall be designed for a force of 50 lbs. per square foot on one and one-half the vertical projection of the structure unloaded; or 30 lbs. per square foot on the same surface, plus 400 lbs. per lineal foot of structure applied 7 ft. above the rail for assumed wind load on trains when the structure is either fully loaded or loaded on either track with empty cars, assumed to weigh 1,200 lbs. per lineal foot, whichever gives the larger stress.

§ 15. **Longitudinal Force.**—Longitudinal bracing for viaduct towers and similar structures shall be designed for a longitudinal force, applied at the rail, of 20 per cent of the live load.

§ 16. **Centrifugal Force of Train.**—Structures located on curves shall be designed for the centrifugal force of the live load acting at the top of the rail. The centrifugal force shall be calculated by the following formula:

$$C = (0.043 - 0.003D)W.D.$$

where $C =$ centrifugal force in lbs.
$W =$ weight of train in lbs.
$D =$ degree of curvature.

The wind loads, as given in Cooper's Specifications for Steel Railway Bridges and Viaducts, 1906 edition, are as follows:

§ 24. **Wind Loads.**—To provide for wind stresses and vibrations from high speed trains, the top lateral bracing in deck bridges, and the bottom lateral bracing in through bridges, shall be proportioned to resist a lateral force of 600 lbs. for each foot of the span; 450 lbs. of this to be treated as a moving load, and as acting on a train of cars, at a line 6 feet above base of rail.

The bottom lateral bracing in deck bridges, and the top lateral bracing in through bridges, shall be proportioned to resist a lateral force of 200 lbs. for each lineal foot for spans up to 300 ft., and 10 lbs. additional for each additional 30 ft.

§ 25. In trestle towers the bracing and columns shall be proportioned to resist the following lateral forces, in addition to the stresses from dead and live loads:

First.—With either one track loaded with cars only, or with both tracks loaded with maximum train load, the lateral forces specified in § 24; and a lateral force of 100 lbs. for each vertical lineal foot of the trestle bents; or

Second.—With both tracks unloaded, a lateral force of 500 lbs. for each longitudinal lineal foot of the structure, acting at the center line of the girders; and a lateral force of 200 lbs. for each vertical lineal foot of the trestle bents.

§ 26. For determining the requisite anchorage for a loaded structure, the train shall be assumed to weigh 1,100 lbs. per lineal foot.

The wind loads, as given in the American Bridge Co.'s Specifications of Steel Railway Bridges, 1901 edition, are as follows:

§ 16. **Wind Loads.**—The wind pressure shall be assumed acting in either direction horizontally:

First.—At 30 lbs. per square foot on the exposed surface of all trusses and the floor as seen in elevation, in addition to a train of 10 ft. average height, beginning 2 ft. 6 ins. above base of rail, moving across the bridge.

Second.—At 50 lbs. per square foot on the exposed surface of all trusses and the floor system. The greatest result shall be assumed in proportioning the parts.

§ 17. For determining the requisite anchorage for the loaded structure, the train shall be assumed to weigh 800 lbs. per lineal foot.

SNOW LOAD.—Snow load is usually not considered separately. In localities where the snow is heavy the snow load should be taken into account. Loose and packed snow may be assumed to weigh 5 and 12 lbs. per cubic foot, respectively.

CHAPTER III.

METHODS FOR THE CALCULATION OF STRESSES IN FRAMED STRUCTURES.

Introduction.—Structures are acted upon by external forces, called loads, the weight of the structure, the reactions of the supports, the force of the wind, etc. These external forces are held in equilibrium by internal forces called stresses. If a straight member is acted upon at its ends by two equal external forces in the direction of its length, equilibrium at any right section of the member will be maintained by internal forces called stresses acting on opposite sides of the section, equal in amount, but opposite in direction to the external forces. When the external forces tend to elongate the member, the stress is tension; when the external forces tend to shorten the member, the stress is compression; while when the external forces tend to shear the member off, the stress is shear. Strain is the deformation caused by stress; the ratio of stress to strain being equal to a quantity, usually a constant, called the modulus of elasticity. Compressive stresses will be considered as positive stresses, while tensile stresses will be considered as negative stresses.

Forces acting in a plane are called coplanar forces. Coplanar forces alone will be considered in this chapter. Forces meeting in a common point are called concurrent forces, while forces which do not all meet in a common point are called non-concurrent forces.

Representation of Forces.—A force is determined when its magnitude, line of action and direction are known. It may be represented algebraically by stating the number of units in the force, by giving the coördinates of a point in the line of action of the force, and by stating the angle made by the line of action of the force with a line of reference; or it may be represented graphically in magnitude by the length of a line, in line of action by the position of the line, and in direction

by an arrow placed on the line pointing in the direction in which the force acts.

Equilibrium.—Statics considers forces at rest, and therefore in equilibrium. To have static equilibrium in any system of forces there must be neither translation nor rotation, and the following conditions must be fulfilled for coplanar forces.

$$\Sigma \text{ horizontal components of forces} = 0 \qquad (1)$$
$$\Sigma \text{ vertical components of forces} = 0 \qquad (2)$$
$$\Sigma \text{ moments of forces about any point} = 0 \qquad (3)$$

Problems in statics can be solved graphically or algebraically. The determination of the reactions of a simple framed structure usually requires the use of equations (1), (2) and (3). Having completely determined the external forces the internal stresses may be obtained by the use of equations (1) and (2) (resolution), or equation (3) (moments). These equations may be solved by algebra or graphics.

There are, therefore, four methods of calculating stresses, viz.:

$$\text{Resolution of Forces}\begin{cases}\text{Algebraic Method.}\\ \text{Graphic Method.}\end{cases}$$
$$\text{Moments of Forces}\begin{cases}\text{Algebraic Method.}\\ \text{Graphic Method.}\end{cases}$$

The stresses in any simple framed structure can be calculated by using any one of the four methods. However, there is usually one method best suited to the solution of each particular problem.

RESOLUTION.—In calculating the stresses in a truss by resolution the fundamental equations for equilibrium for translation

$$\Sigma \text{ horizontal components of forces} = 0 \qquad (1)$$
$$\Sigma \text{ vertical components of forces} = 0 \qquad (2)$$

are applied to the structure at the joints or to sections.

Force Triangle.—The resultant, R, of the two forces P_1 and P_2 meeting at the point a in Fig. 50 is represented in magnitude and direction by the diagonal, R, of the parallelogram a–b–c–d. The combining of the two forces P_1 and P_2 into the force R is termed composition of forces. The reverse process is called resolution of forces.

The value of R may be found algebraically from the equation

$$R^2 = P_1^2 + P_2^2 + 2P_1P_2 \cos \theta$$

It is not necessary to construct the entire force parallelogram as in (a) Fig. 50, the force triangle (b) below or (c) above the resultant R being sufficient.

If only one force together with the line of action of the two others be given in a system containing three forces in equilibrium, the magnitude and direction of the two forces may be found by means of the force triangle.

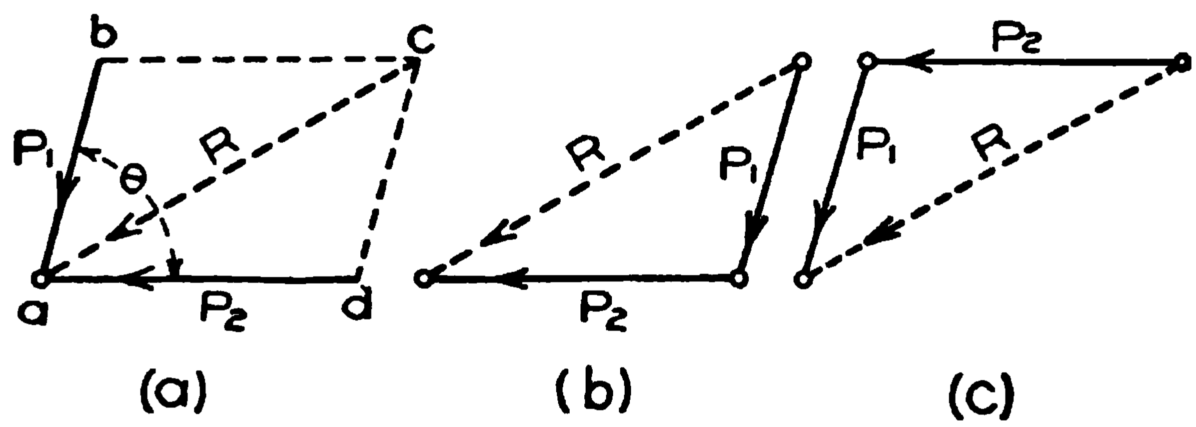


FIG. 50.

If the resultant R in Fig. 50 is replaced by a force E equal in amount but opposite in direction, the system of forces will be in equilibrium, (a) or (b) Fig. 51. The force E is the equilibrant of the system of forces P_1 and P_2.

It is immaterial in what order the forces are taken in constructing the force triangle, as in Fig. 51, as long as the forces all act in the same direction around the triangle. The force triangle is the foundation of the science of graphic statics.

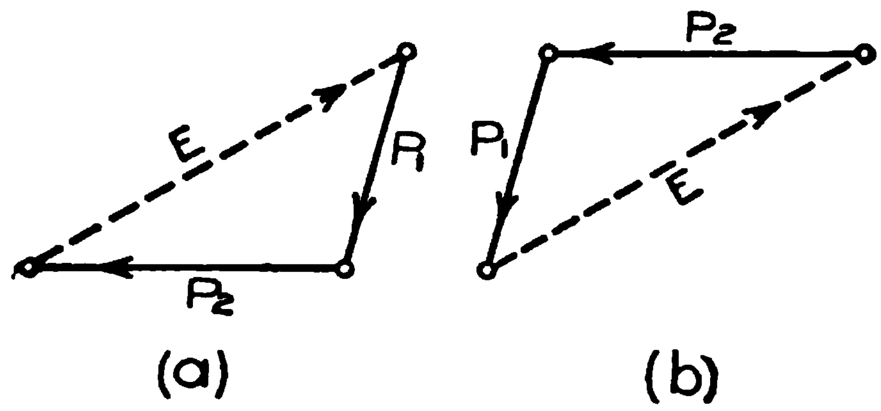


FIG. 51.

Force Polygon.—If more than three concurrent forces (forces which meet in a point) are in equilibrium as in (a) Fig. 52, R_1 in (b) will be the resultant of P_1 and P_2, R_2 will be the resultant of R_1 and P_3,

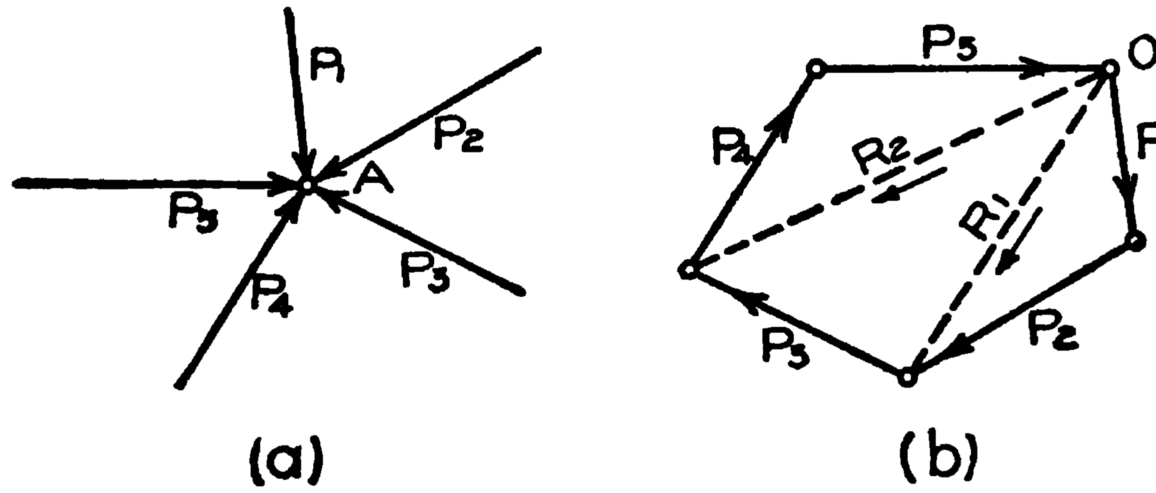


FIG. 52.

and will also be the equilibrant of P_4 and P_5. The force polygon in (b) is therefore only a combination of force triangles. The force polygon for any system of forces may be constructed as follows: Beginning at any point draw in succession lines representing in magnitude and direction the given forces, each line beginning where the preceding one ends. If the polygon closes, the system of forces is in equilibrium, if it does not close the line joining the first and last points represents the resultant in magnitude and direction. As in the case of the force triangle, it is immaterial in what order the forces are applied as long as they all act in the same direction around the polygon. A force polygon is analogous to a traverse of a field in which the bearings and the distances are measured progressively around the field in either direction. The conditions for closure in the two cases are also identical.

It will be seen that any side in the force polygon is the equilibrant of all the other sides, and that any side reversed in direction is the resultant of all the other sides.

Equilibrium of Concurrent Forces.—The necessary condition for equilibrium of concurrent coplaner forces therefore is that the force polygon close. This is equivalent to the algebraic condition that Σ horizontal components of forces $= 0$, and Σ vertical components of forces $= 0$. If the system of concurrent forces is not in equilibrium,

the resultant can be found in magnitude and direction by completing the force polygon. The resultant of a system of concurrent forces is always a single force acting through their point of intersection.

Algebraic Resolution.—In calculating the stresses in a truss by algebraic resolution, the fundamental equations for equilibrium, (1) and (2), for translation are applied (a) to each joint, or (b) to the members and forces on one side of a section cut through the truss.

(a) *Forces at a Joint.*—The reactions having been found, the stresses in the members of the truss shown in Fig. 53 are calculated as

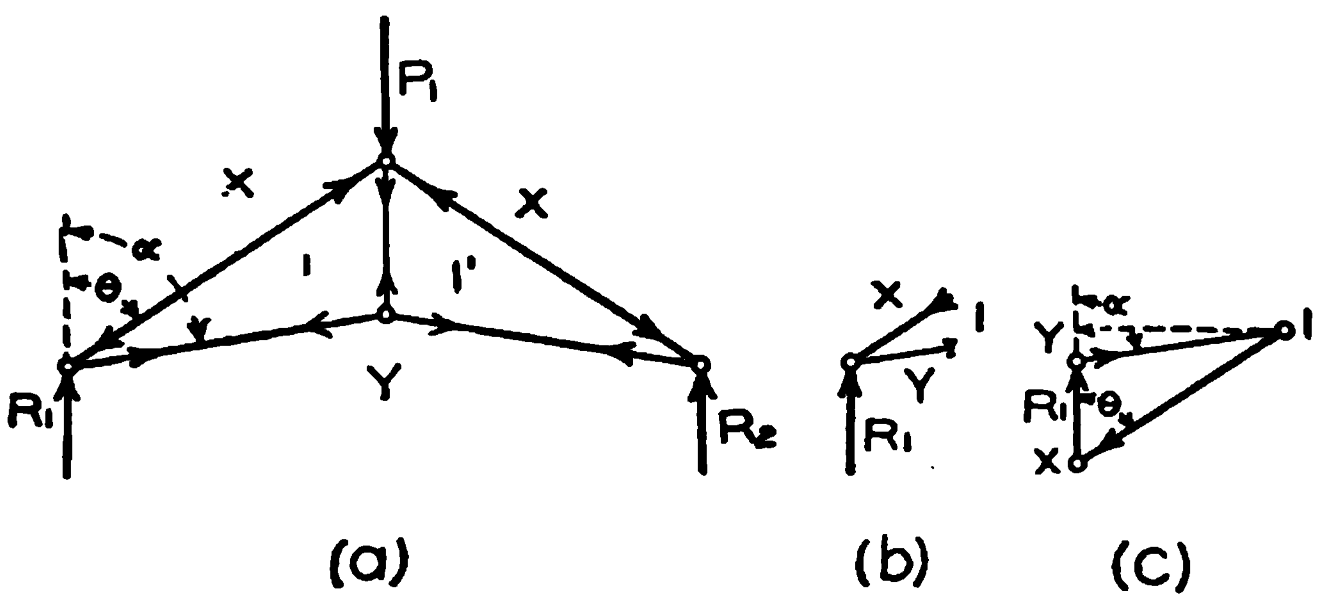


FIG. 53.

follows: Beginning at the left reaction, R_1, we have by applying equations (1) and (2)

$$1\text{-}x \sin\theta - 1\text{-}y \sin\alpha = 0 \qquad (4)$$

$$1\text{-}x \cos\theta - 1\text{-}y \cos\alpha - R_1 = 0 \qquad (5)$$

The stresses in members 1-x and 1-y may be obtained by solving equations (4) and (5). The direction of the forces which represent the stresses in amount will be determined by the signs of the results; if compressive stresses are assumed as positive, tensile stresses will be negative. Arrows pointing toward the joint indicate that the member is in compression; arrows pointing away from the joint indicate that the member is in tension. The stresses in the members of the truss at the remaining joints in the truss are calculated in the same way.

The direction of the forces and the kind of stress can always be

determined by sketching in the force polygon, for the forces meeting at the joint as in (c) Fig. 53.

It will be seen from the foregoing that the method of algebraic resolution consists in applying the principle of the force polygon to the external forces and internal stresses at each joint.

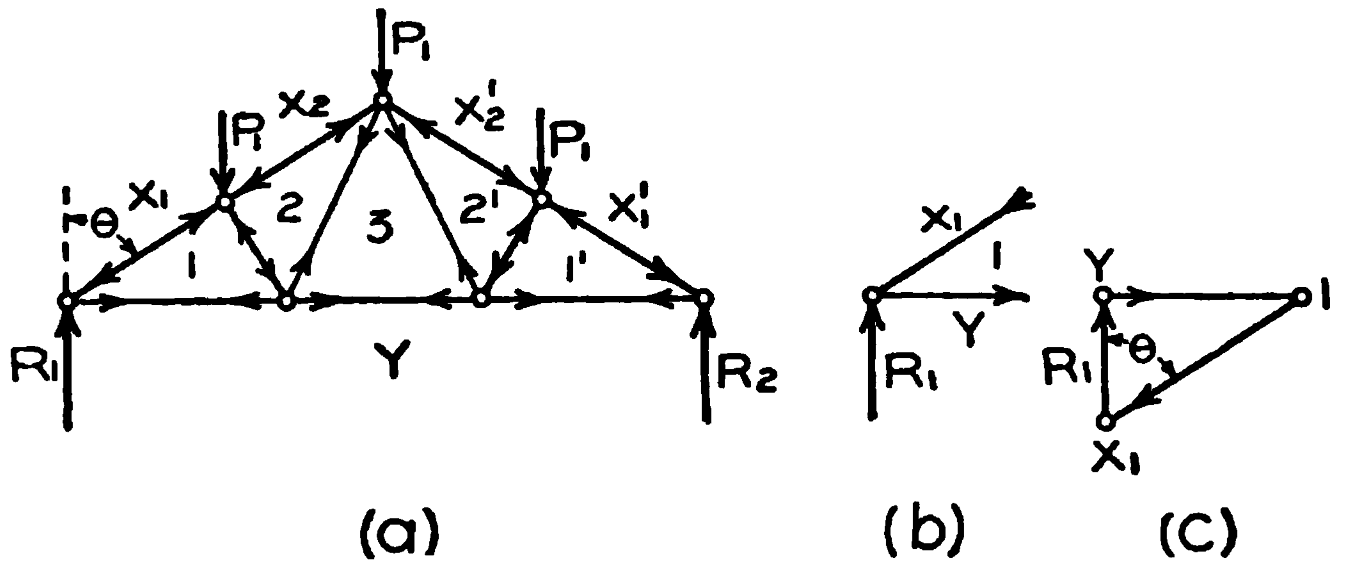


FIG. 54.

Since we have only two fundamental equations for translation (resolution) we can not solve a joint if there are more than two forces or stresses unknown.

Where the lower chord of the truss is horizontal as in Fig. 54, we have by applying fundamental equations (1) and (2) to the joint at the left reaction

$$1\text{-}x = + R_1 \sec\theta \tag{6}$$

$$1\text{-}y = - R_1 \tan\theta \tag{7}$$

the plus sign indicating compression and the minus sign tension. Equations (6) and (7) may be obtained directly from force triangle (c).

(b) *Forces on One Side of a Section.*—The principle of resolution of forces may be applied to the structure as a whole or to a portion of the structure.

If the truss shown in Fig. 55 is cut by the plane *A-A*, the internal stresses and external forces acting on either segment, as in (b) will be in equilibrium. The external forces acting on the cut members as shown in (b) are equal to the internal stresses in the cut members and are opposite in direction.

Applying equations (1) and (2) to the cut section

$$3\text{-}y + 2\text{-}3 \cos \alpha - 2\text{-}x \sin \theta = 0 \qquad (8)$$

$$2\text{-}3 \sin \alpha - 2\text{-}x \cos \theta + R_1 - P_1 = 0 \qquad (9)$$

Now, if all but two of the external forces are known, the unknowns may be found by solving equations (8) and (9). If more than two

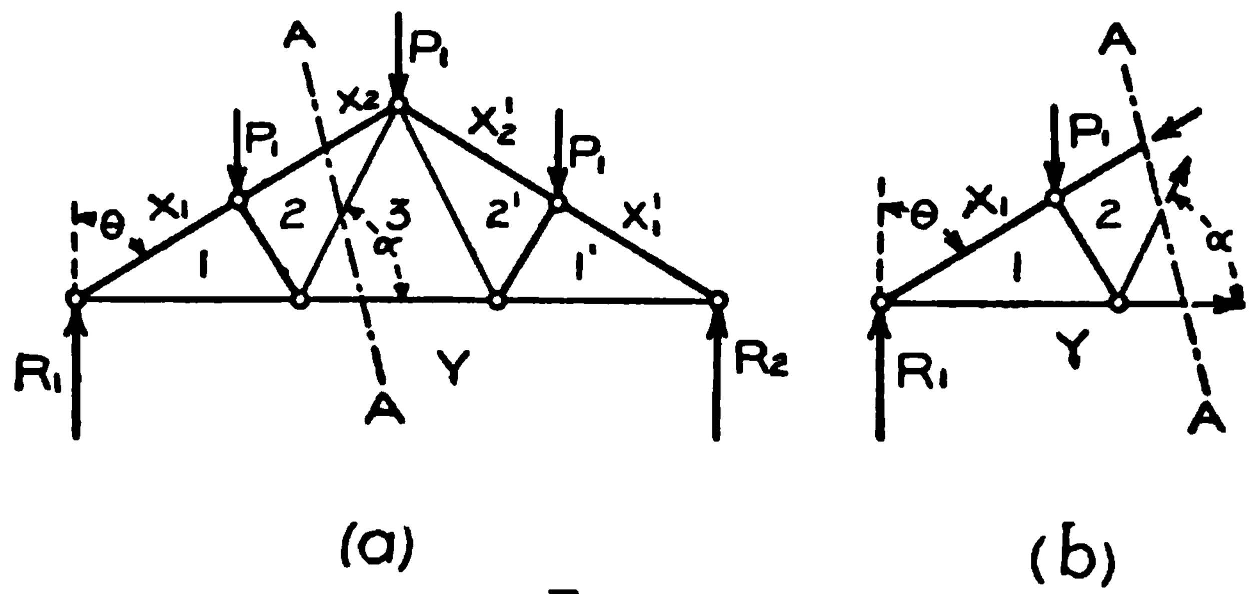


FIG. 55.

external forces are unknown the problem is indeterminate as far as equations (8) and (9) are concerned.

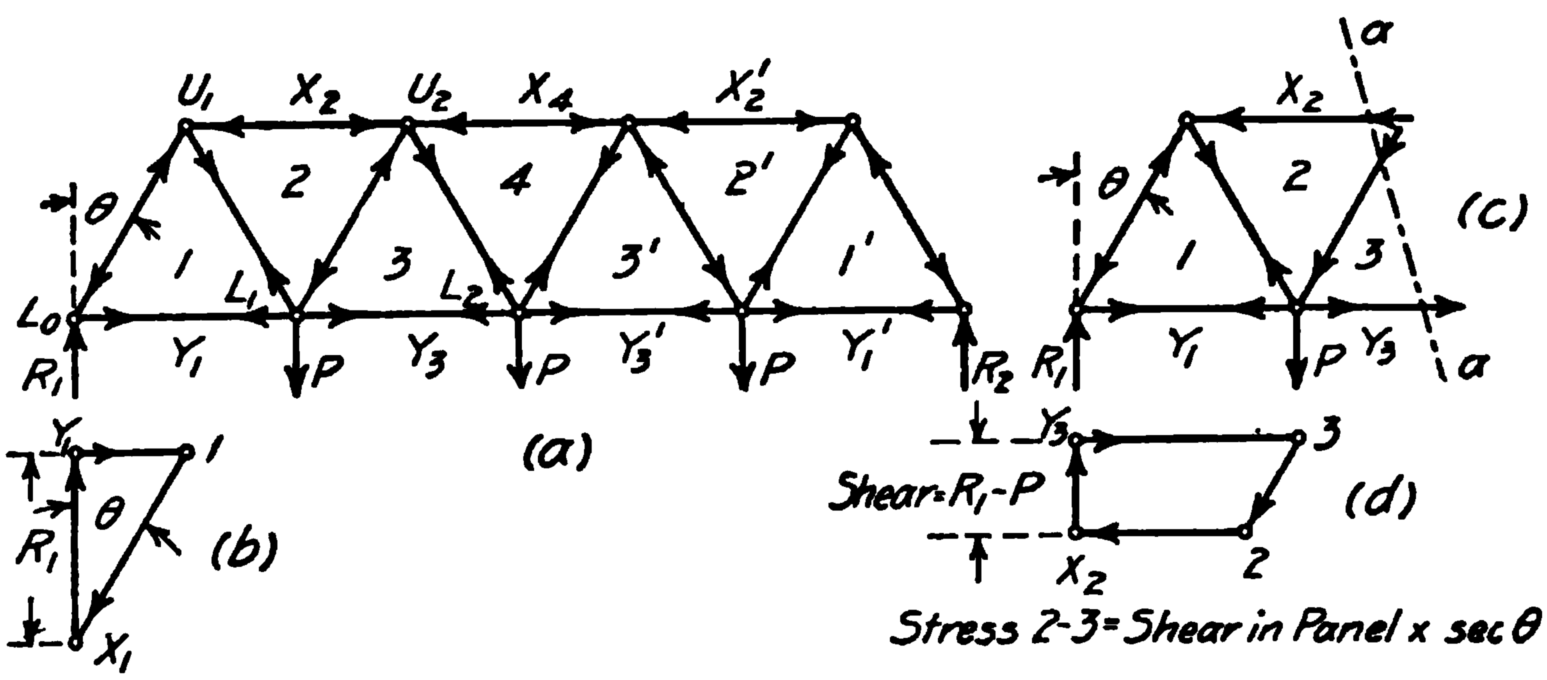


FIG. 56.

In the Warren truss in Fig. 56 the stresses at a joint may be calculated by completing the force polygon as at the left reaction in (b) Fig. 54. Applying equations (1) and (2) to a section as in (c)

$$2\text{-}x + 2\text{-}3 \sin \theta - 3\text{-}y = 0 \qquad (10)$$

$$-2\text{-}3 \cos \theta - P + R_1 = 0 \qquad (11)$$

Now, $R_1 - P =$ shear in the panel. Therefore the stress in 2–3 $= -(R_1 - P) \sec\theta =$ shear in panel $\times \sec\theta$. This analysis leads directly to the method of coefficients as explained in detail in Chapter V.

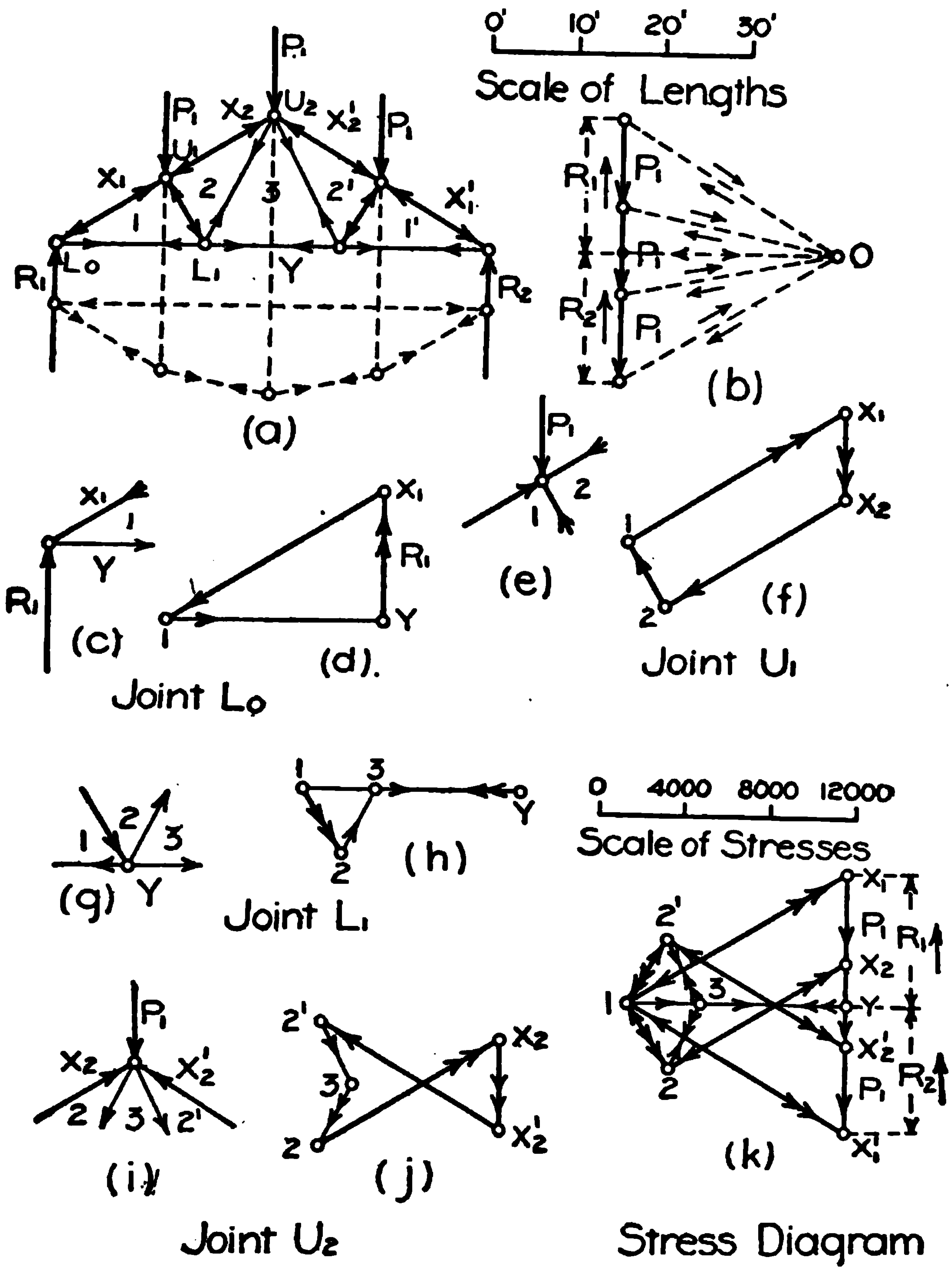


FIG. 57.

Graphic Resolution.—In Fig. 57 the reactions R_1 and R_2 are found by means of the force and equilibrium polygons as shown in (b) and (a). The principle of the force polygon is then applied to each joint of the structure in turn. Beginning at the joint L_0 the forces

are shown in (c), and the force triangle in (d). The reaction R_1 is known and acts upward, the upper chord stress 1–x acts downward to the left, and the lower chord stress 1–y acts to the right closing the polygon. Stress 1–x is compression and stress 1–y is tension, as can be seen by applying the arrows to the members in (c). The force polygon at joint U_1 is then constructed as in (f). Stress 1–x acting toward joint U_1 and load P_1 acting downward are known, and stresses 1–2 and 2–x are found by completing the polygon. Stresses 2–x and 1–2 are compression. The force polygons at joints L_1 and U_2 are constructed, in the order given, in the same manner. The known forces at any joint are indicated in direction in the force polygon by double arrows, and the unknown forces are indicated in direction by single arrows.

The stresses in the members of the right segment of the truss are the same as in the left, and the force polygons are, therefore, not constructed for the right segment. The force polygons for all the joints of the truss are grouped into the stress diagram shown in (k). Compression in the stress diagram and truss is indicated by arrows acting toward the ends of the stress lines and toward the joints, respectively, and tension is indicated by arrows acting away from the ends of the stress lines and away from the joints, respectively. The first time a stress is used a single arrow, and the second time the stress is used a double arrow is used to indicate direction. It will be seen that the upper chords are in compression, while the lower chord is in tension. The stress diagram in (k) Fig. 57 is called a "Maxwell diagram" or a "reciprocal polygon diagram."

The notation used is known as Bow's notation, in which points in the truss diagram become areas in the stress diagram, and areas in the truss diagram become points in the stress diagram. The method of graphic resolution is the method most commonly used for calculating stresses in roof trusses and simple framed structures with inclined chords.

For the analysis of the stresses in roof trusses, see the author's book, "The Design of Steel Mill Buildings."

Warren Bridge Truss.—In Fig. 58 the dead load stresses in a Warren bridge truss loaded on the lower chord, are calculated by the method of graphic resolution. In the stress diagram the loads are laid off from the bottom upwards. The details of the solution can

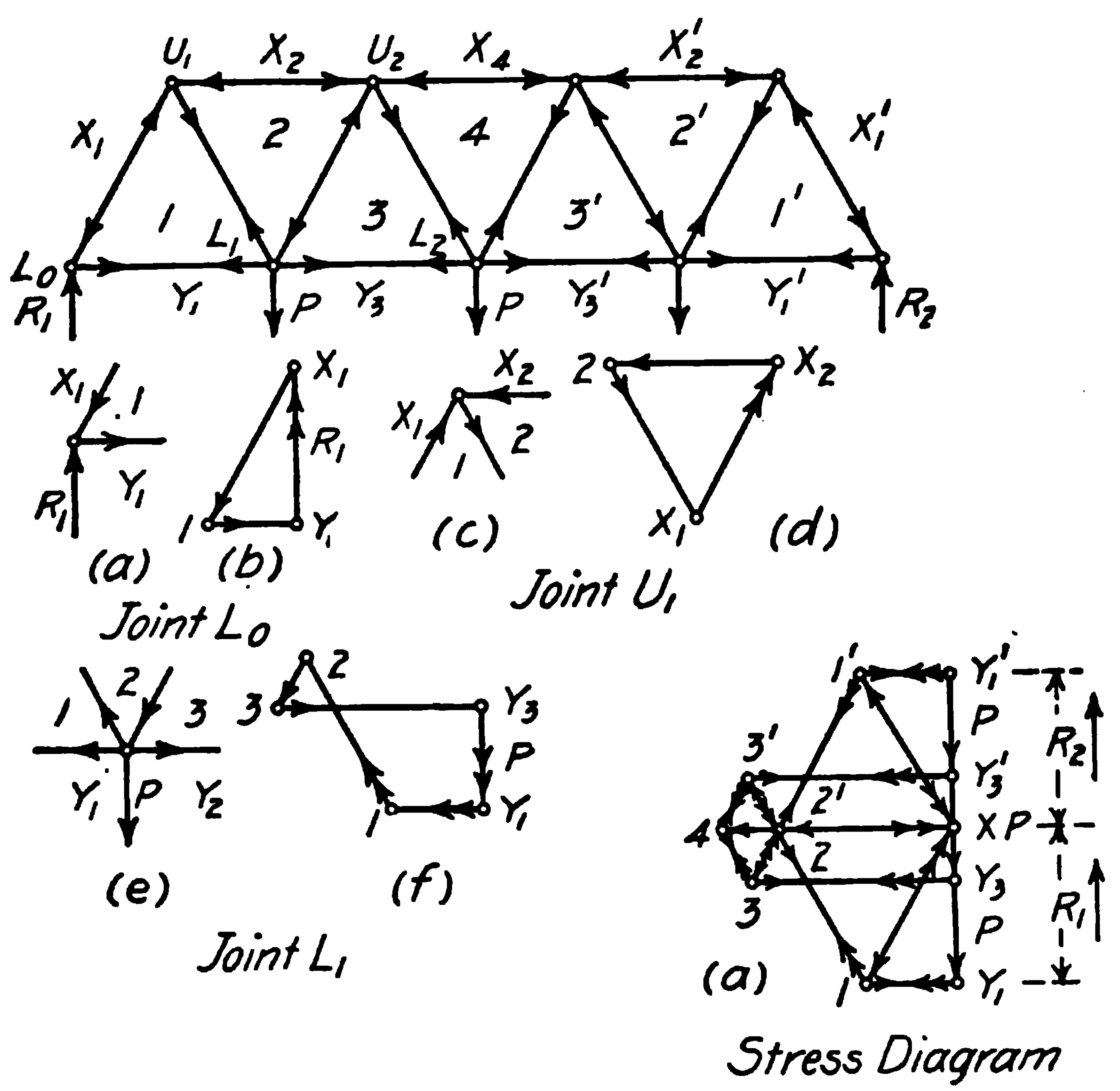


Fig. 58.

easily be followed by reference to Fig. 58 and Fig. 57. It will be seen that the upper chord of the truss is in compression, while the lower chord is in tension.

MOMENTS.—In calculating the stresses in a truss by moments, the fundamental equation for equilibrium for rotation

$$\Sigma \text{ moments about any point} = 0 \qquad (3)$$

is applied to parts of the structure. Equation (3) may be solved either by algebra or by graphics. Before applying equation (3) to the parts of a structure it will be necessary to discuss a few fundamental principles.

Equilibrium of Non-concurrent Forces.—If the forces are non-concurrent (do not all meet in a common point), the condition that the force polygon close is a necessary, but not a sufficient condition for equilibrium. For example, take the three equal forces P_1, P_2 and P_3, making an angle of 120° with each other as in (a) Fig. 59.

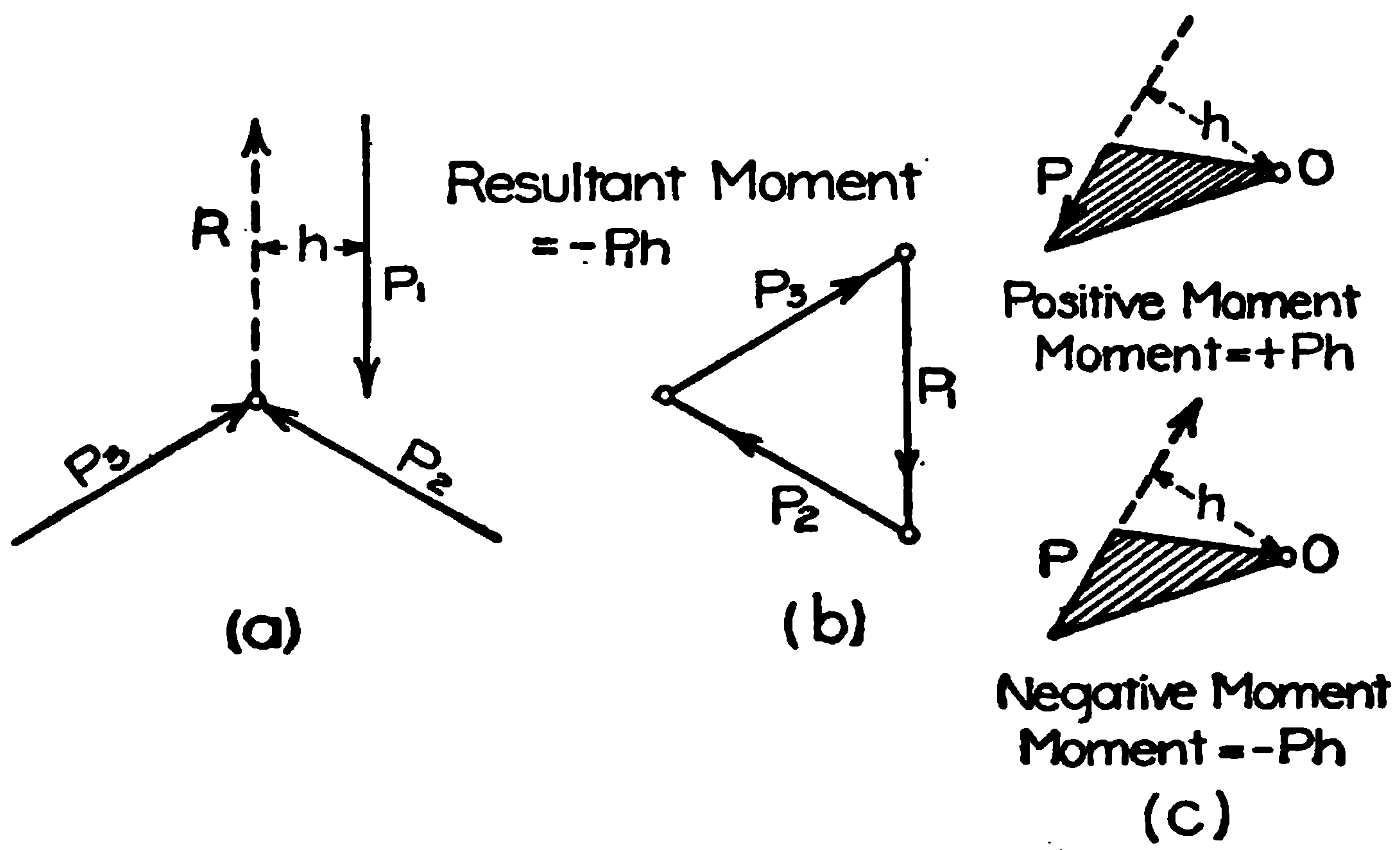


FIG. 59.

The force polygon (b) closes, but the system is not in equilibrium. The resultant, R, of P_2 and P_3 acts through their intersection and is parallel to P_1, but is opposite in direction. The system of forces is in equilibrium for translation, but is not in equilibrium for rotation.

The resultant of this system is a couple with a moment $= -P_1 \cdot h$, moments clockwise being considered negative and counter-clockwise positive, (c) Fig. 59. The equilibrant of the system in (a) Fig. 59 is a couple with a moment $= + P_1 \cdot h$.

A couple.—A couple consists of two parallel forces equal in amount, but opposite in direction. The arm of the couple is the perpendicular distance between the forces. The moment of a couple is equal to one of the forces multiplied by the arm. The moment of a couple is constant about any point in the plane and may be represented graphically by twice the area of the triangle having one of the forces as a base and the arm of the couple as an altitude. The moment of a force about

any point may be represented graphically by twice the area of a triangle, as shown in (c) Fig. 59.

It will be seen from the preceding discussion, that in order that a system of non-concurrent forces be in equilibrium it is necessary that the resultant of all the forces save one shall coincide with the one and be opposite in direction. Three non-concurrent forces can not be in equilibrium unless they are parallel. The resultant of a system of non-concurrent forces may be a single force or a couple.

Equilibrium Polygon. *First Method.*—In Fig. 60 the resultant, a, of P_1 and P_2 acts through their intersection and is equal and parallel to a in the force polygon (a); the resultant, b, of a and P_3 acts through

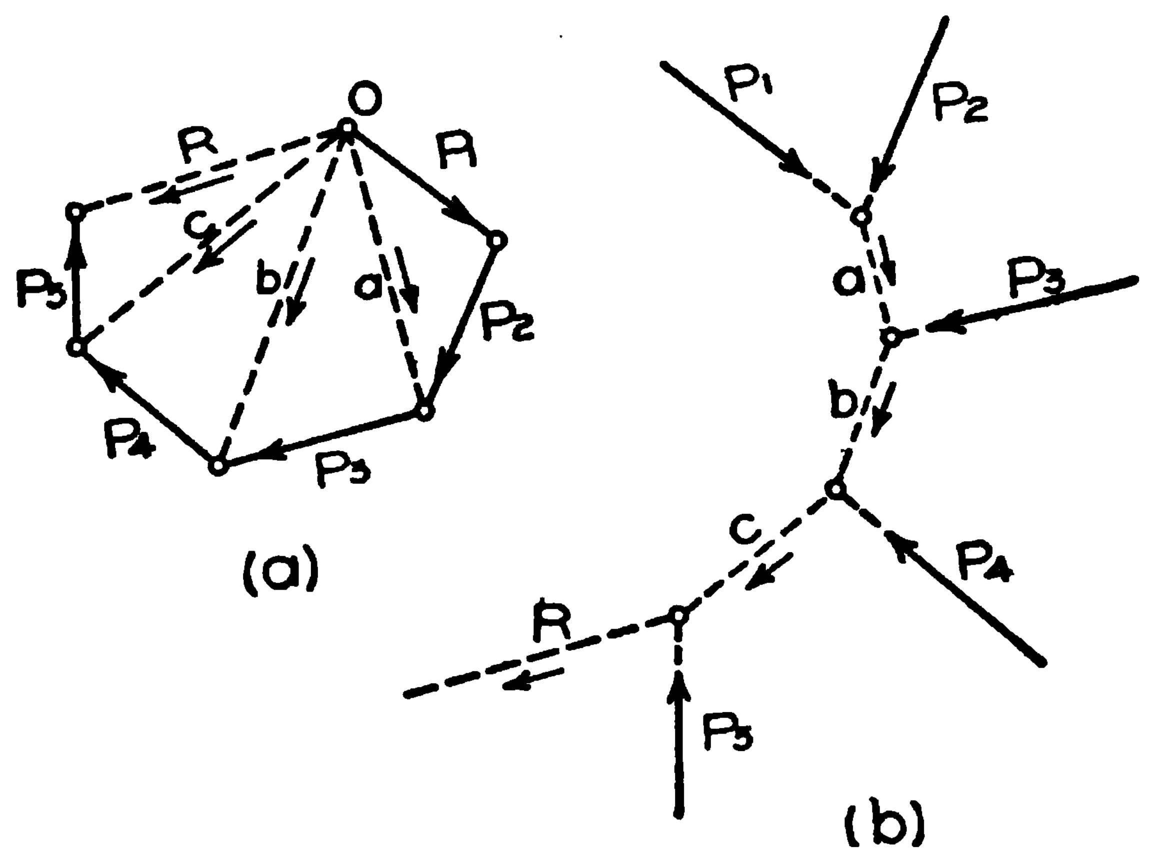


FIG. 60.

their intersection and is equal and parallel to b in the force polygon; the resultant, c, of b and P_4 acts through their intersection and is equal and parallel to c in the force polygon; and finally the resultant, R, of c and P_5 acts through their intersection and is equal and parallel to R in the force polygon. R is therefore the resultant of the entire system of forces. If R is replaced by an equal and opposite force, E, the system of forces will be in equilibrium. Polygon (a) in Fig. 60 is called a force polygon and (b) is called a "funicular" or an "equilibrium" polygon. It will be seen that the magnitude and direction of the resul-

tant of a system of forces is given by the closing line of the force polygon, and the line of action is given by the equilibrium polygon.

The force polygon in (a) Fig. 61 closes and the resultant, R, of the forces P_1, P_2, P_3, P_4, P_5 is parallel and equal to P_6, and is opposite

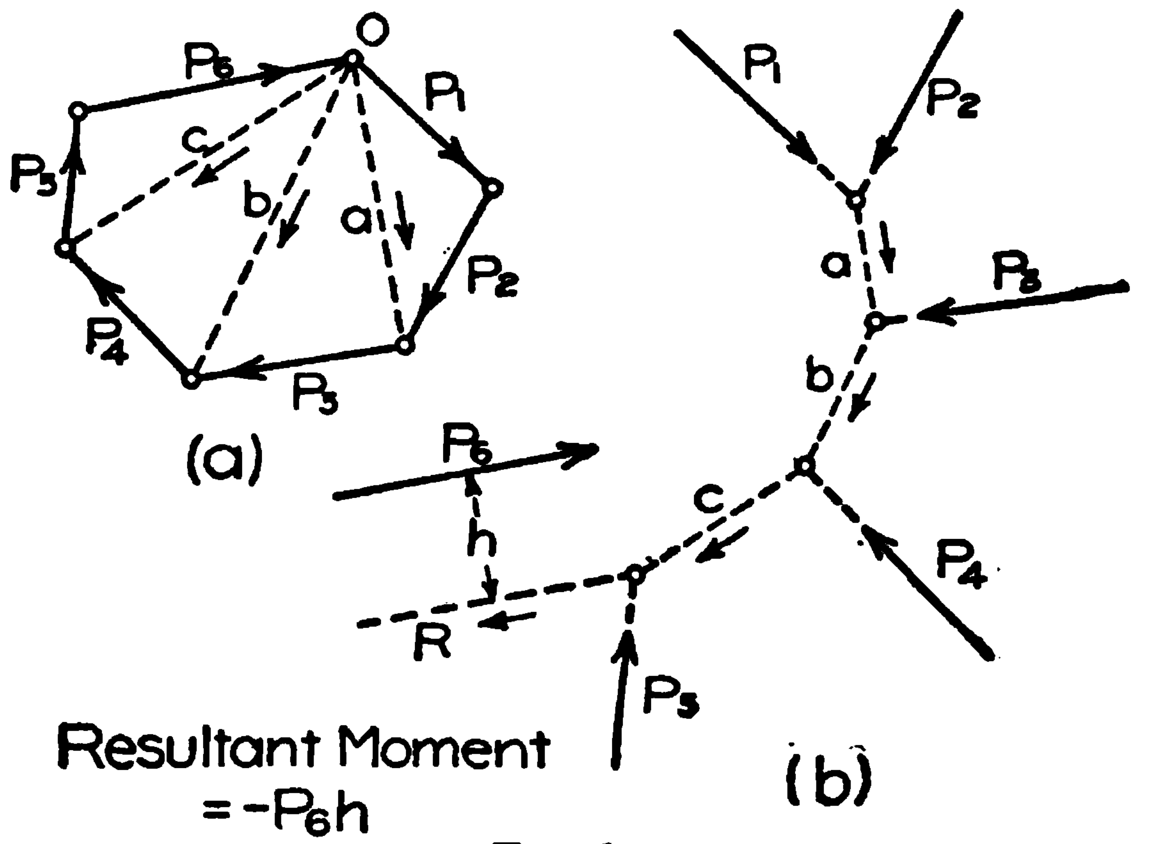


FIG. 61.

in direction. The system is in equilibrium for translation, but is not in equilibrium for rotation. The resultant is a couple with a moment $= -P_6 \cdot h$. The equilibrant of the system of forces will be a couple with a moment $= + P_6 \cdot h$. From the preceding discussion it will be seen that if the force polygon for any system of non-concurrent forces closes the resultant will be a couple. If there is perfect equilibrium the arm of the couple will be zero.

Second Method.—Where the forces do not intersect within the limits of the drawing board, or where the forces are parallel, it is not possible to draw the equilibrium polygon as shown in Fig. 60 and Fig. 61, and the following method is used:

The point o, (a) Fig. 62, which is called the pole of the force polygon, is selected so that the strings $a\text{–}o$, $b\text{–}o$, $c\text{–}o$, $d\text{–}o$ and $e\text{–}o$ in the equilibrium polygon (b), which are drawn parallel to the corresponding rays in the force polygon (a), will make good intersections with the forces which they replace or equilibrate.

In the force polygon (a), P_1 is equilibrated by the imaginary forces

represented by the rays *o–a* and *b–o*, acting as indicated by the arrows within the triangle; P_2 is equilibrated by the imaginary forces represented by the rays *o–b* and *c–o*, acting as indicated by the arrows within the triangle; P_3 is equilibrated by the imaginary forces represented by the rays *o–c* and *d–o*, acting as indicated by the arrows within the tri-

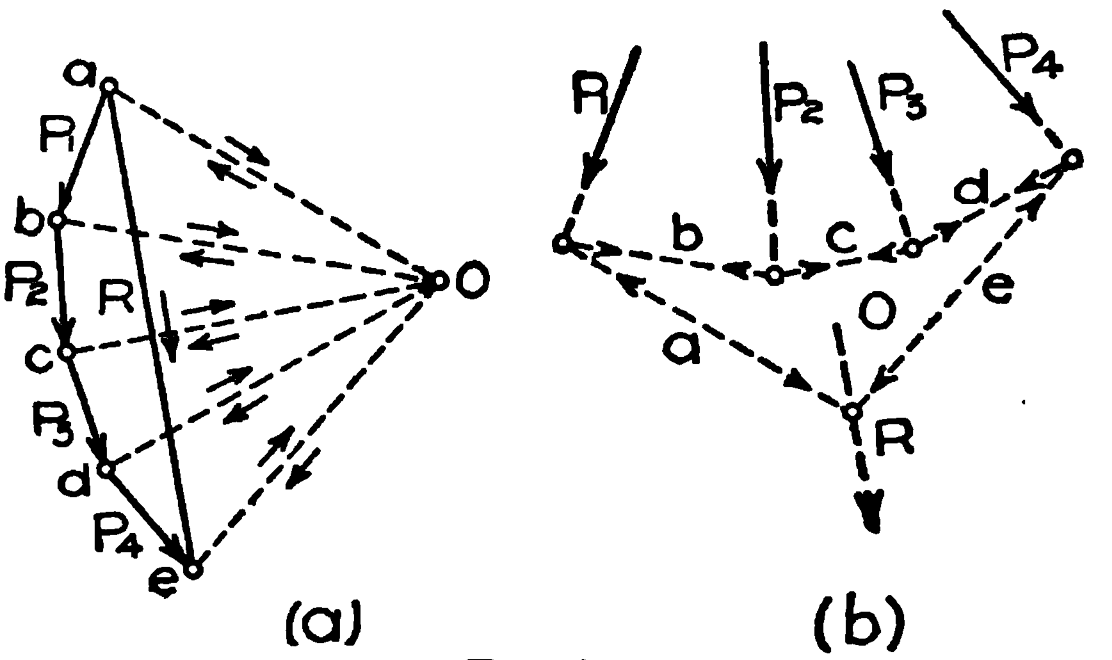


FIG. 62.

angle; and P_4 is equilibrated by the imaginary forces *o–d* and *e–o*, acting as indicated by the arrows within the triangle. The imaginary forces are all neutralized except *a–o* and *o–e*, which are seen to be components of the resultant, *R*.

To construct the equilibrium polygon, take any point on the line of action of P_1 and draw strings *o–a* and *o–b* parallel to rays *o–a* and *o–b*, *b–o* is the equilibrant of *o–a* and P_1; through the intersection of string *o–b* and P_2 draw string *c–o* parallel to ray *c–o*, *c–o* is the equilibrant of *o–b* and P_2; through the intersection of string *c–o* and P_3 draw string *d–o* parallel to ray *d–o*, *d–o* is the equilibrant of *c–o* and P_3; and through the intersection of string *d–o* and P_4 draw string *e–o* parallel to ray *e–o*, *e–o* is the equilibrant of *d–o* and P_4. Strings *o–a* and *e–o* acting as shown are components of the resultant, *R*, which will be parallel to *R* in the force polygon and acts through the intersections of strings *o–a* and *e–o*.

The imaginary forces represented by the rays in the force polygon may be considered as components of the forces and the analysis made on that assumption with equal ease.

It is immaterial in what order the forces are taken in drawing the force polygon, as long as the forces all act in the same direction around the force polygon, and the strings meeting on the lines of the forces in the equilibrium polygon are parallel to the rays drawn to the ends of the same forces in the force polygon.

The imaginary forces *a–o, b–o, c–o, d–o, e–o* are represented in magnitude and in direction by the rays of the force polygon to the same scale as the forces P_1, P_2, P_3, P_4. The strings of the equilibrium polygon represent the imaginary forces in line of action and direction, but not in magnitude.

Graphic Moments.—In Fig. 63 (b) is a force polygon and (a) is an equilibrium polygon for the system of forces P_1, P_2, P_3, P_4.

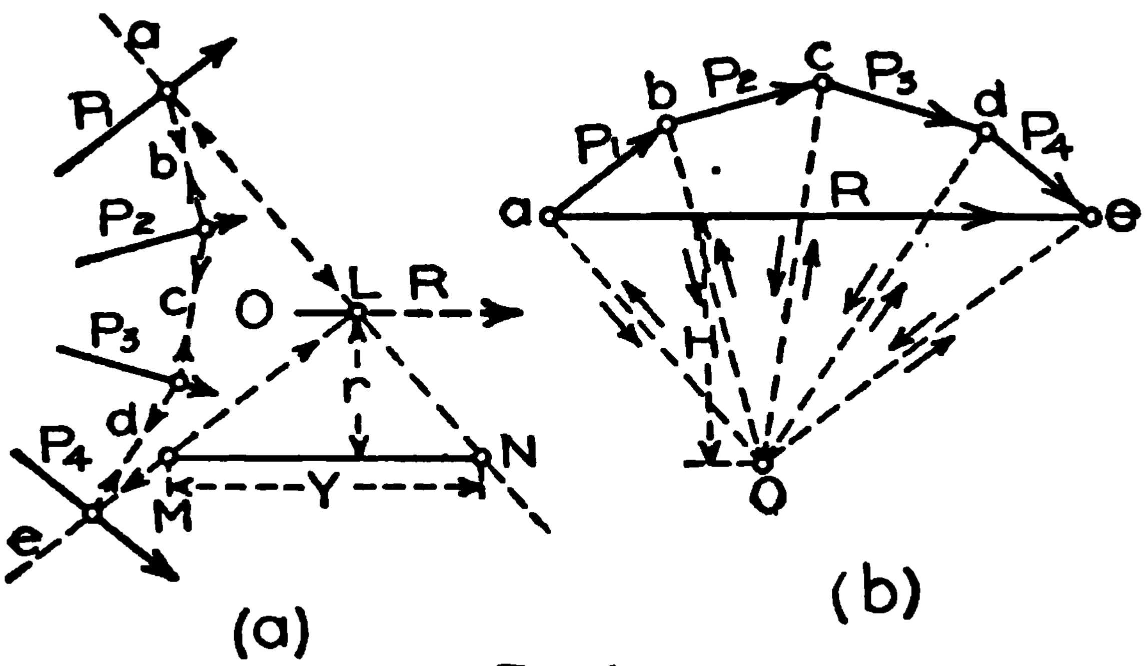


FIG. 63.

Draw the line $M–N = y$, parallel to the resultant, R, and with ends on strings *o–e* and *o–a* produced. Let r equal the altitude of the triangle $L–M–N$, and H equal the altitude of the similar triangle *o–e–a*. H is the pole distance of the resultant, R.

Now, in the similar triangles $L–M–N$ and *o–e–a*

$$R : y :: H : r$$

and

$$R \cdot r = H \cdot y$$

But $R \cdot r = M =$ moment of resultant R about any point in the line $M–N$ and therefore

$$M = H \cdot y \tag{12}$$

The statement of the principle just demonstrated is as follows: *The moment of any system of coplanar forces about any point in the plane is equal to the intercept on a line drawn through the center of moments and parallel to the resultant of all the forces, cut off by the strings which meet on the resultant, multiplied by the pole distance of the resultant.* It should be noted that in all cases the intercept is a distance and the pole distance is a force.

This property of the equilibrium polygon is frequently used in calculating the bending moments in beams and trusses which are loaded with vertical loads.

Bending Moments in a Beam.—It is required to find the moment at the point M in the simple beam loaded as in (b) Fig. 64. The

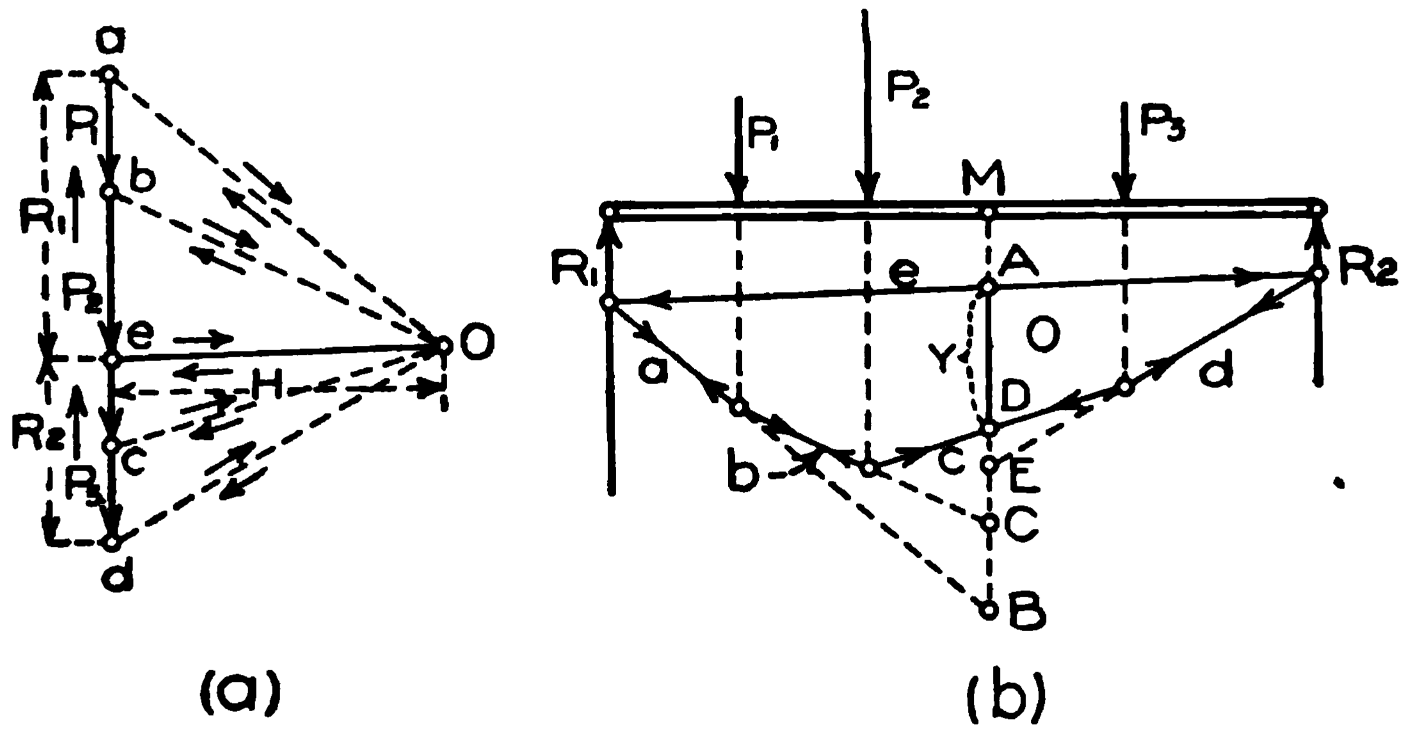


FIG. 64.

moment at M will be the algebraic sum of the moments of the forces to the left of M. The moment of $P_1 = H \times B\text{–}C$, the moment of $P_2 = H \times C\text{–}D$ and the moment of $R_1 = -H \times B\text{–}A$. The moment at M will therefore be

$$M_1 = H \times B\text{–}C + H \times C\text{–}D - H \times B\text{–}A = -H \times A\text{–}D = -H \cdot y$$

The moment of the forces to the right of M may in like manner be shown to be

$$M_1 = + H \cdot y$$

In like manner the bending moment at any point in the beam may be shown to be the ordinate of the equilibrium polygon multiplied by the pole distance. The ordinate is a distance and is measured by the same scale as the beam, while the pole distance is a force and is measured by the same scale as the loads.

Equilibrium Polygon as a Framed Structure.—In (a) Fig 65 the rigid triangle supports the load P_1. Construct a force polygon by

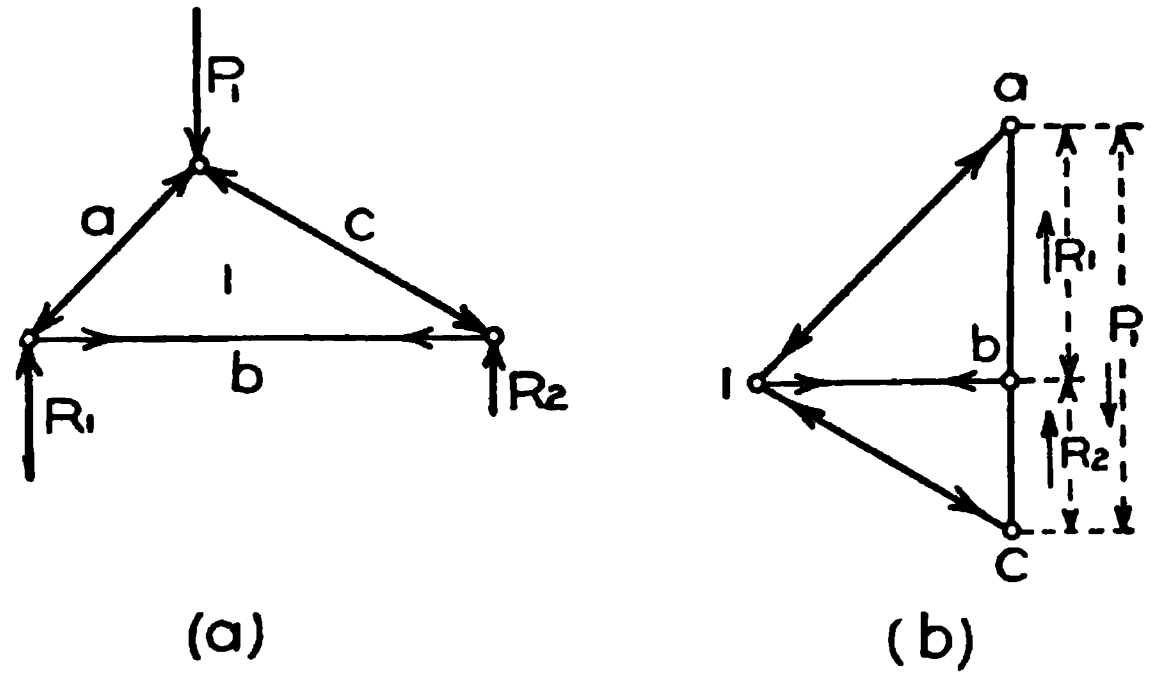


FIG. 65.

drawing rays a–1 and c–1 in (b) parallel to sides a–1 and c–1, respectively, in (a), and through pole 1 draw 1–b parallel to side 1–b in (a). The reactions R_1 and R_2 will be given by the force polygon (b), and the rays 1–a, 1–c and 1–b represent the stresses in the members 1–a, 1–c and 1–b, respectively, in the triangular structure. The stresses in 1–a and 1–c are compression and the stress in 1–b is tension, forces acting toward the joint indicating compression and forces acting away from the joint indicating tension. Triangle (a) is therefore an equilibrium polygon, and polygon (b) is a force polygon for the force P_1.

From the preceding discussion it will be seen that the internal stresses at any point or in any section hold in equilibrium the external forces meeting at the point, or on either side of the section.

Algebraic Moments. Stresses in a Roof Truss.—The reactions may be found by applying the fundamental equations of equilibrium to the structure as a whole. In the truss in (a) Fig. 66 by taking moments about the right reaction we have

$$R_1 \times 6d = 5P \times 3d$$

$$R_1 = {}^5\!/_2 P_1 = R_2$$

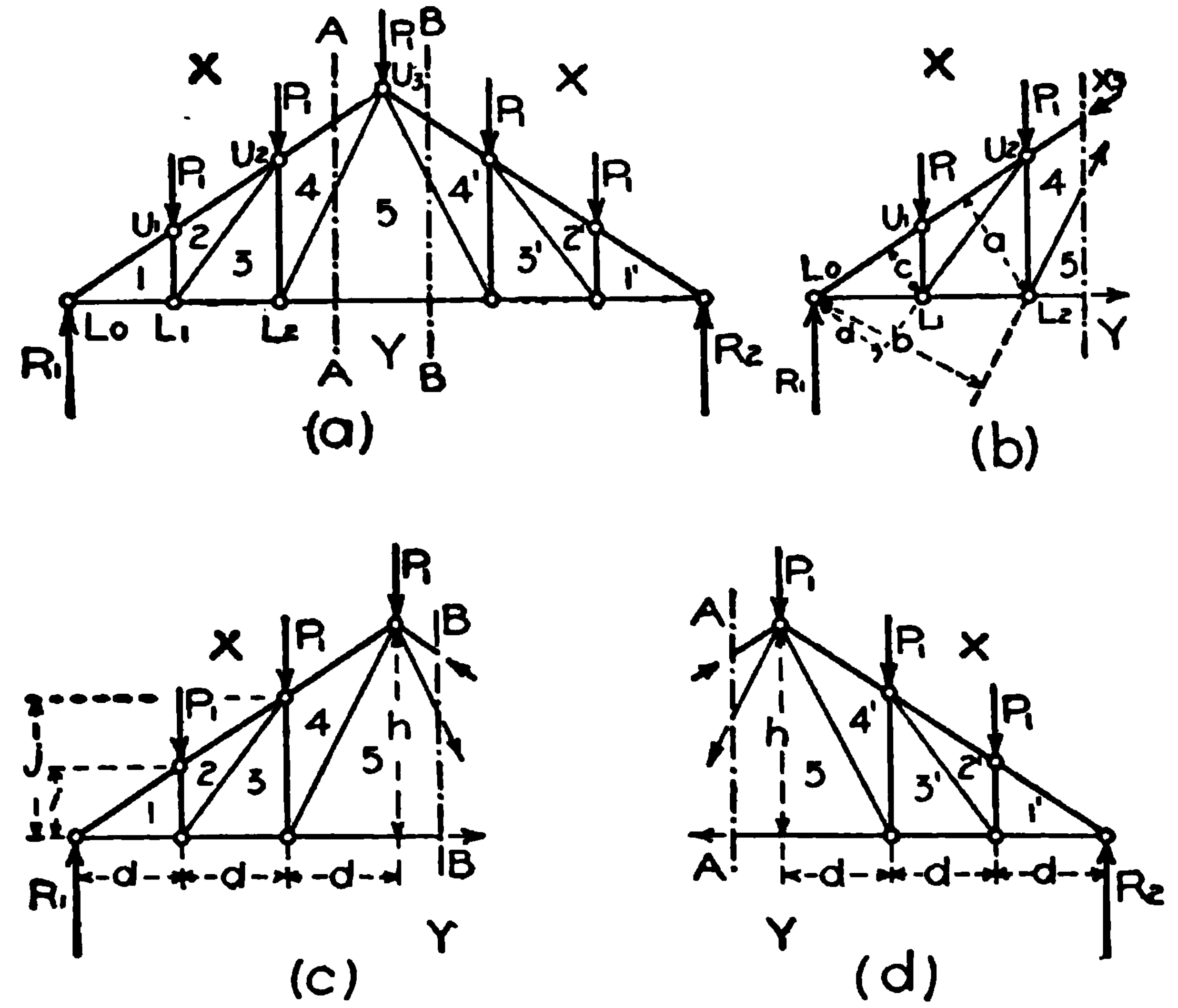


FIG. 66.

To find the stresses in the members of the truss in (a) Fig. 66, proceed as follows: Cut the truss by means of plane A–A, as in (b), and replace the stresses in the members cut away with external forces. These forces are equal to the stresses in the members in amount, but opposite in direction, and produce equilibrium.

To obtain stress 4–x take center of moments at L_2, and take moments of external forces

$$4\text{-}x \times a + P_1 \times d - R_1 \times 2d = 0$$

$$4\text{-}x = \frac{R_1 \times 2d - P_1 \cdot d}{a} = \frac{4P \cdot d}{a} \text{ (compression)}$$

To obtain stress in 4–5 take center of moments at L_0, and take moments of external forces

$$4\text{-}5 \times b - 2P_1 \times {}^3\!/_2 d = 0$$

$$4\text{-}5 = \frac{3P_1 \cdot d}{b} \text{ (tension)}$$

To obtain the stress in 5–y take center of moments at joint U_3 in (c), and take moments of external forces

$$5\text{-}y \times h - R_1 \times 3d + 3P_1 \cdot d = 0$$

$$5\text{-}y = \frac{3R_1 \cdot d - 3P_1 \cdot d}{h} = \frac{9P_1 \cdot d}{2h} \text{ (tension)}$$

To Determine Kind of Stress.—If the unknown external force is always taken as acting from the outside toward the cut section, *i. e.*, is always assumed to cause compression, the sign of the result will indicate the kind of stress. A plus sign will indicate that the assumed direction was correct and that the stress is compression, while a minus sign will indicate that the assumed direction was incorrect and that the stress is tension.

In calculating stresses by algebraic moments, therefore, always observe the following rule:

Assume the unknown external force as acting from the outside toward the cut section; a plus sign for the result will then show that the stress in the member is compression, and a minus sign will indicate that the stress in the member is tension.

The stresses in the web members 3–4, 2–3, 1–2, are found by taking moments about joint L_0 as a center. The stresses in y–3 and y–1 are found by taking moments about joints U_2 and U_1, respectively; and the stresses in x–2 and x–1 are found by taking moments about joint L_1.

The method of algebraic moments is the most common method used for calculating the stresses in bridge trusses with inclined chords, and similar frameworks which carry moving loads.

Stresses in a Bridge Truss.—Calculate reaction R_1 by taking moments of the vertical forces about joints L_0'. Then $R_1 \times L = 6P \cdot L/2$, and $R_1 = 3P = R_2$. To calculate the stress in any member in the truss, pass a section cutting the member in which the stress is required, and cutting away the truss on one side of the section. The stresses in the

members cut away are assumed as replaced by external forces acting in the line of the member and equal to the stresses in amount.

To calculate the stresses take the center of moments so that there will be but one unknown stress. The solution of the equation of moments about this center of moments will give the required stress. To calculate the stress in 4–5 in (b) Fig. 67, pass the section *a–a*, cutting away the right side of the truss, and take the center of moments at the intersection of the top and bottom chords. Now 5–*x* and 4–*y*

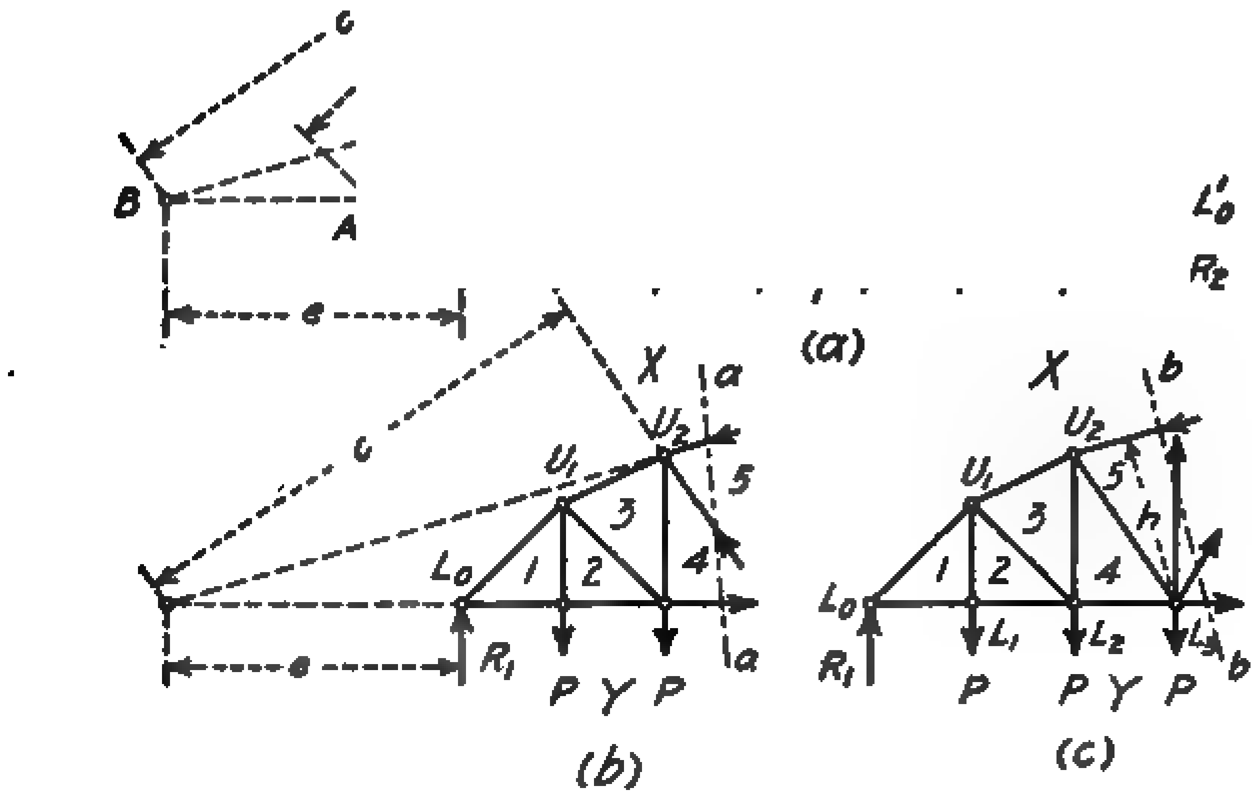


FIG. 67.

act through the center of moments and produce no moment. The moment of the stress in 4–5 acting from the outside toward the cut section with an arm *c*, holds in equilibrium the reaction R_1, and the two loads, *P*. The sign of the result will determine the kind of stress, minus for tension and plus for compression. To calculate the stress in the top chord U_2U_3, pass section *b–b* in (c) and take moments about joint L_3.

Graphic Moments.—The bending moment at any point in a truss may be found by means of a force and equilibrium polygon as in (b) and (a) Fig. 68. To determine the stress in 4–x, cut section A–A and take moments about joint L_2 as in Fig. 68. The moment of the exter-

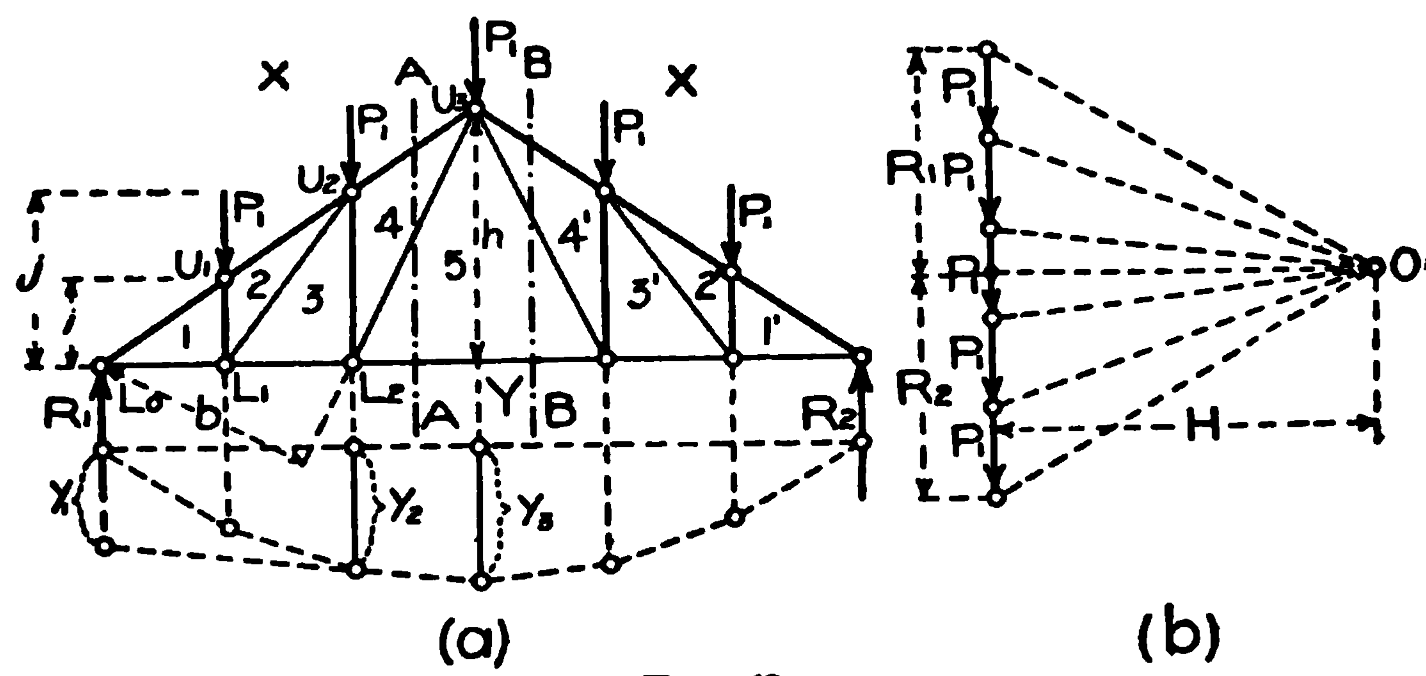


FIG. 68.

nal forces on the left of L_2 will be $M_2 = -H \cdot y_2$, and stress

$$4\text{-}x = -M_2/a = +H \cdot y_2/a$$

To obtain stress in 4–5 take center of moments at joint L_0, and stress

$$4\text{-}5 = M_1/b = -H \cdot y_1/b$$

To obtain stress in 5–y take center of moments at joint U_3, and stress

$$5\text{-}y = M_3/h = -H \cdot y_3/h$$

The method of graphic moments is principally used to explain other methods and is little used as a direct method of calculation.

CHAPTER IV.

Stresses in Beams.

Introduction.—Simple and cantilever beams, only, will be considered in this chapter. For the calculation of stresses in continuous beams, see the author's "Steel Mill Buildings," Chapter XVa.

Reactions of a Simple Beam.—A force and an equilibrium polygon may be used to obtain the reactions of a beam loaded with a load P, as in Fig. 70.

The force polygon (b) is drawn with a pole O at any convenient point, and rays O–a and O–c are drawn. Now from the fundamental

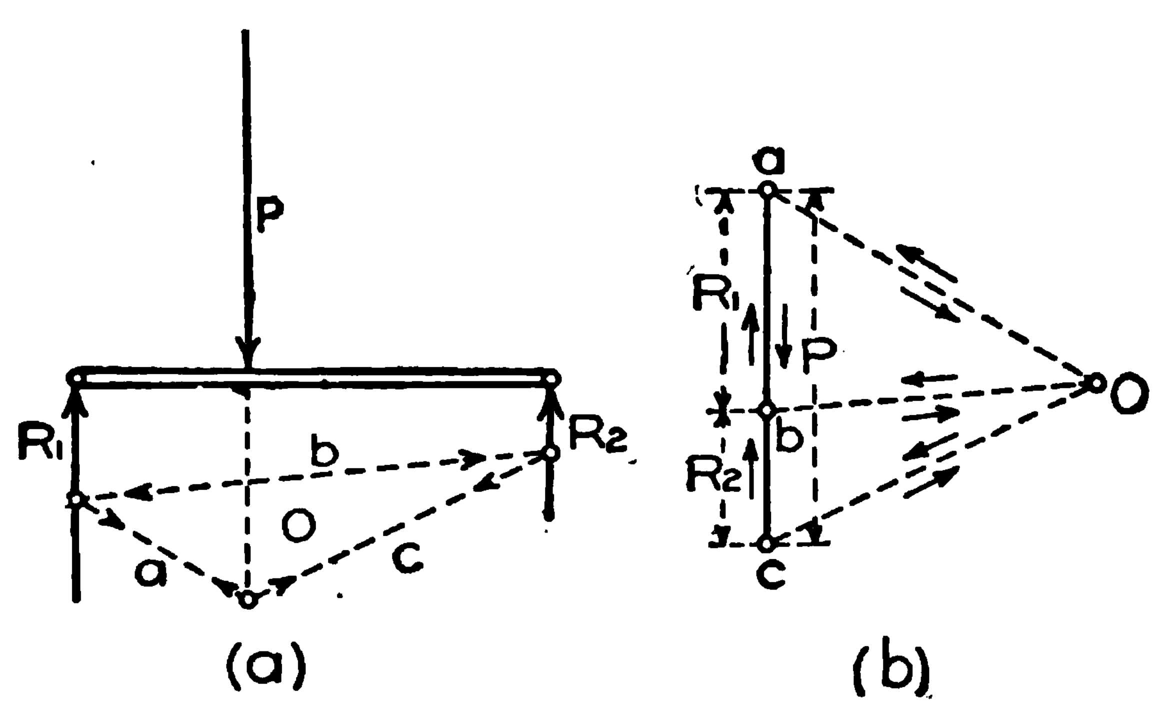


Fig. 70.

conditions for equilibrium for translation we have $P = R_1 + R_2$. At any convenient point in the line of action of P, draw the strings O–a and O–c parallel to the rays O–a and O–c, respectively, in the force polygon. The imaginary forces a–O and O–c acting as shown, equilibrate the force P. The imaginary force a–O acting in a reverse direction, as shown, is an equilibrant of R_1, and the imaginary force c–O, acting in a reverse direction, is an equilibrant of R_2. The remaining equilibrant of R_1 and of R_2 must coincide and be equal in amount, but

opposite in direction. The string b–O is the remaining equilibrant of R_1 and also of R_2, and is called the closing line of the equilibrium polygon. The ray b–O drawn parallel to the string b–O divides P in two parts, which are equal to the reactions R_1 and R_2.

Reactions of a Cantilever Beam.—As a second example let it be required to find the reactions of the overhanging beam shown in Fig. 71.

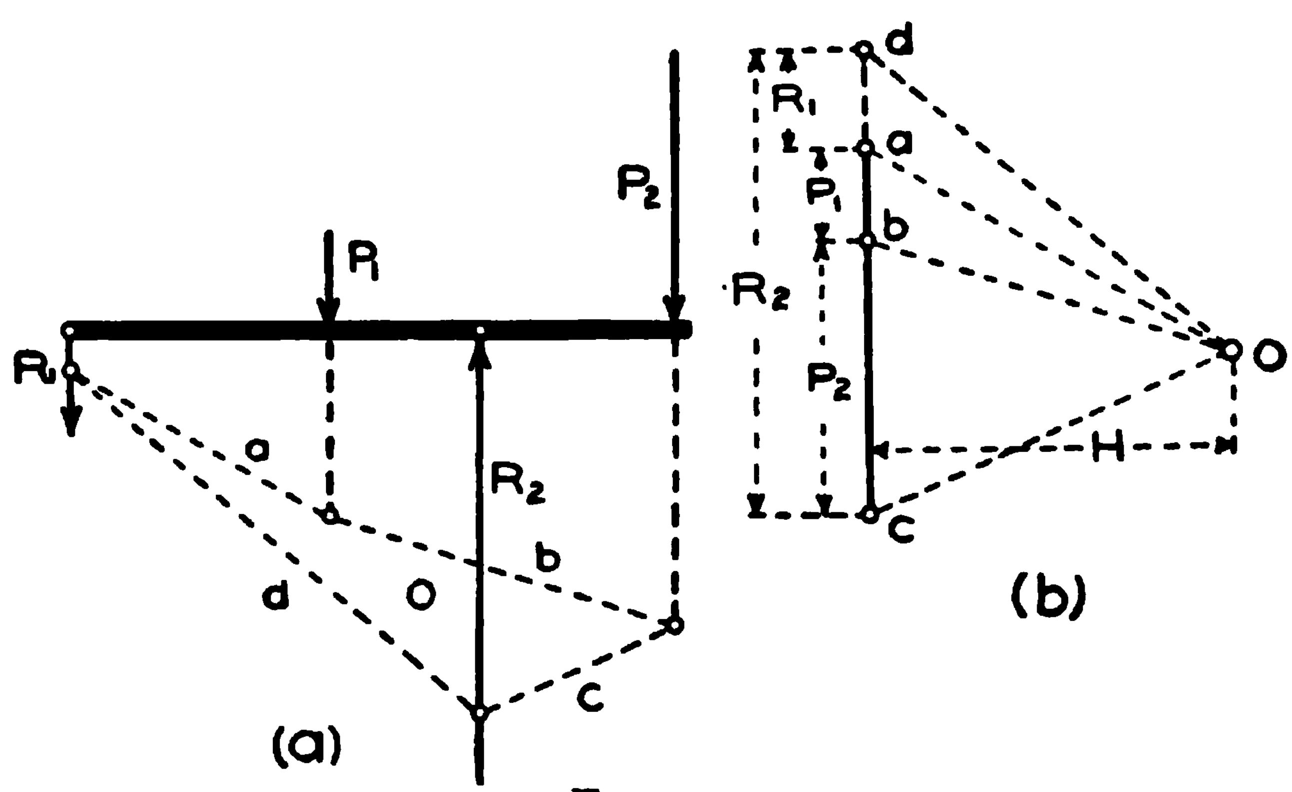


FIG. 71.

Construct a force polygon with pole O, as in (b), and draw an equilibrium polygon, as in (a). The ray O–d, drawn parallel to the closing line O–d in (a), determines the reactions. In this case reaction R_1 is negative. It should be noted that the closing line in an equilibrium polygon must have its ends on the two reactions.

The ordinate to the equilibrium polygon at any point, multiplied by the pole distance, H, will give the bending moment in the beam at a point immediately above it.

Moments and Shears in Beams: Concentrated Loads.—The bending moments in the beam in Fig. 72 may be found by constructing the force polygon (a) and the equilibrium polygon (b) as shown.

The bending moment at any point is then equal to the ordinate to the equilibrium polygon at that point, multiplied by the pole distance, H. The ordinate is to be measured to the same scale as the beam, and the pole distance, H, is to be measured to the same scale as the loads in

the force polygon. The ordinate is a distance and the pole distance is a force.

Or, if the scale to which the beam is laid off be multiplied by the pole distance measured to the scale of the loads, and this scale be used in measuring the ordinates, the ordinates will be equal to the bending moments at the corresponding points. This is the same as making the pole distance equal to unity. Diagram (b) is called a moment diagram.

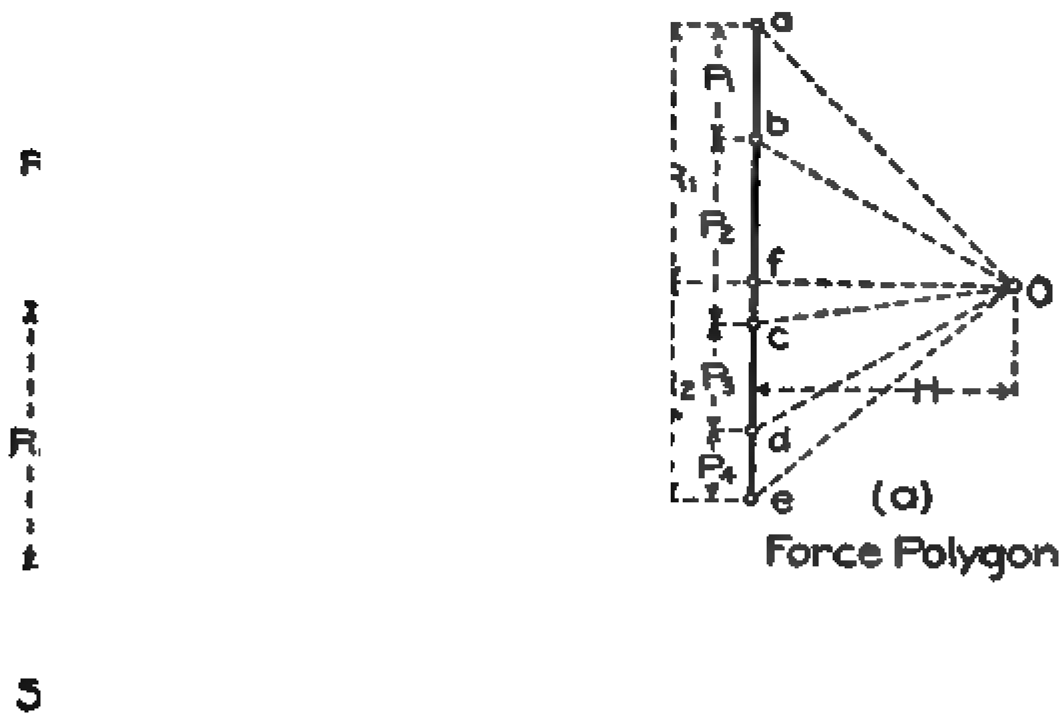


FIG. 72.

Between the left support and the first load the shear is equal to R_1; between the loads P_1 and P_2 the shear equals $R_1 - P_1$; between the loads P_2 and P_3 the shear equals $R_1 - P_1 - P_2$; between the loads P_3 and P_4 the shear equals $R_1 - P_1 - P_2 - P_3$; and between load P_4 and the right reaction the shear equals $R_1 - P_1 - P_2 - P_3 - P_4 = - R_2$. At load P_2 the shear changes from positive to negative. Diagram (c) is called a shear diagram. It will be seen that the maximum ordinate in the moment diagram comes at the point of zero shear.

The bending moment at any point in the beam is equal to the algebraic sum of the shear areas on either side of the point in question.

From this we see that the shear areas on each side of P_2 must be equal. This property of the shear diagram depends upon the principle that *the bending moment at any point in a simple beam is the definite integral of the shear between either point of support and the point in question.* This will be taken up again in the discussion of beams uniformly loaded, which will now be considered.

Moments and Shears in Beams: Uniform Loads.—In the beam loaded with a uniform load of w lbs. per lineal foot shown in Fig. 73, the reaction $R_1 = R_2 = \frac{1}{2}w \cdot L$. At a distance x from the left support, the bending moment is

$$M = R_1 \cdot x - w \cdot x^2/2 = \tfrac{1}{2}w(L \cdot x - x^2) \qquad (13)$$

which is the equation of the common parabola.

The parabola may be constructed by means of the force and equilibrium polygons, by assuming that the uniform load is concentrated at points in the beam, as is assumed in a bridge truss, and then drawing

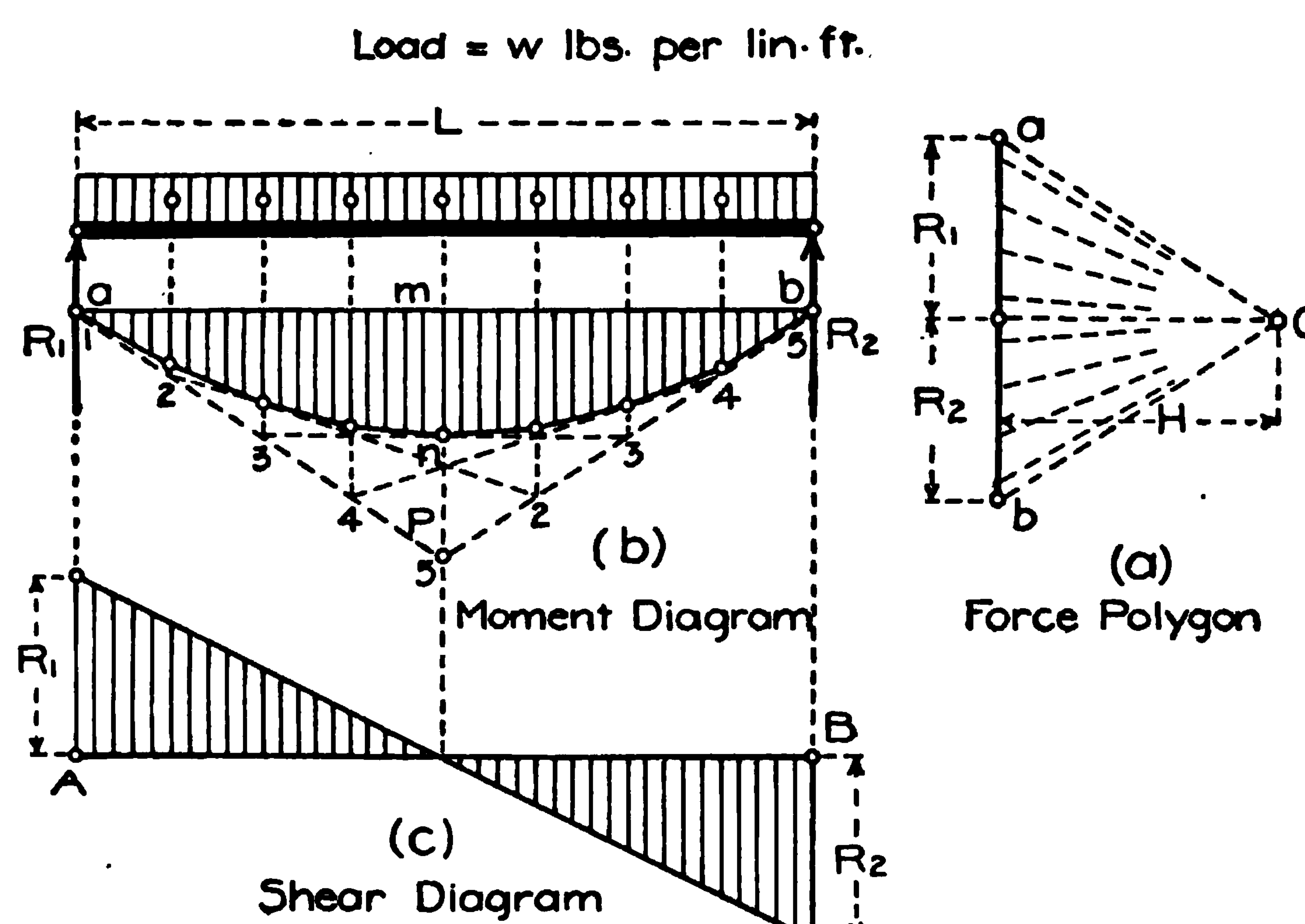


Fig. 73.

the force and equilibrium polygons in the usual way, as in Fig. 73. The greater the number of segments into which the uniform load is

divided, the more nearly will the equilibrium polygon approach the bending moment parabola.

The parabola may be constructed without drawing the force and equilibrium polygons as follows: Lay off ordinate m–n = n–p = bending moment at center of beam = $\frac{1}{8}w \cdot L^2$. Divide a–p and b–p into the same number of equal parts and number them as shown in (b). Join the points with like numbers by lines, which will be tangents to the required parabola. It will be seen in Fig. 73 that points on the parabola are also obtained.

The shear at any point x will be

$$S = R_1 - w \cdot x = \tfrac{1}{2}w \cdot L - w \cdot x = w(L/2 - x) \qquad (14)$$

which is the equation of the inclined line shown in (c) Fig. 73. The shear at any point is therefore represented by the ordinate to the shear diagram at the given point.

Property of the Shear Diagram.—Integrating the equation for shear between the limits, $x = 0$ and $x = x$, we have

$$\int_0^x S dx = \int_0^x w(L/2 - x)dx = \tfrac{1}{2}w\,(L \cdot x - x^2)$$

which is the equation for the bending moment at any point, x, in the beam, and is also the area of the shear diagram between the limits given. From this we see that the bending moment at any point in a simple beam uniformly loaded, is equal to the area of the shear diagram to the left of the point in question. The bending moment is also equal to the algebraic sum of the shear areas on either side of the point.

Beam With Partial Uniform Load.—The beam in Fig. 74 is loaded with a load w extending over a length b. The bending moments between the left end of the uniform load and the left reaction is $R_1 \cdot x$, represented by the ordinates to the straight line A–1 in (a); the bending moments in that part of the beam covered by the uniform load is represented by ordinates to the curved line 1–2; while the bending moments to the right of the uniform load are represented by ordinates to the straight line 2–B. The ordinates from the straight line 1–2 to

the curve 1–2 are the same as for a simple beam with a span b loaded with a uniform load w. The shear diagram is shown in (b). It will

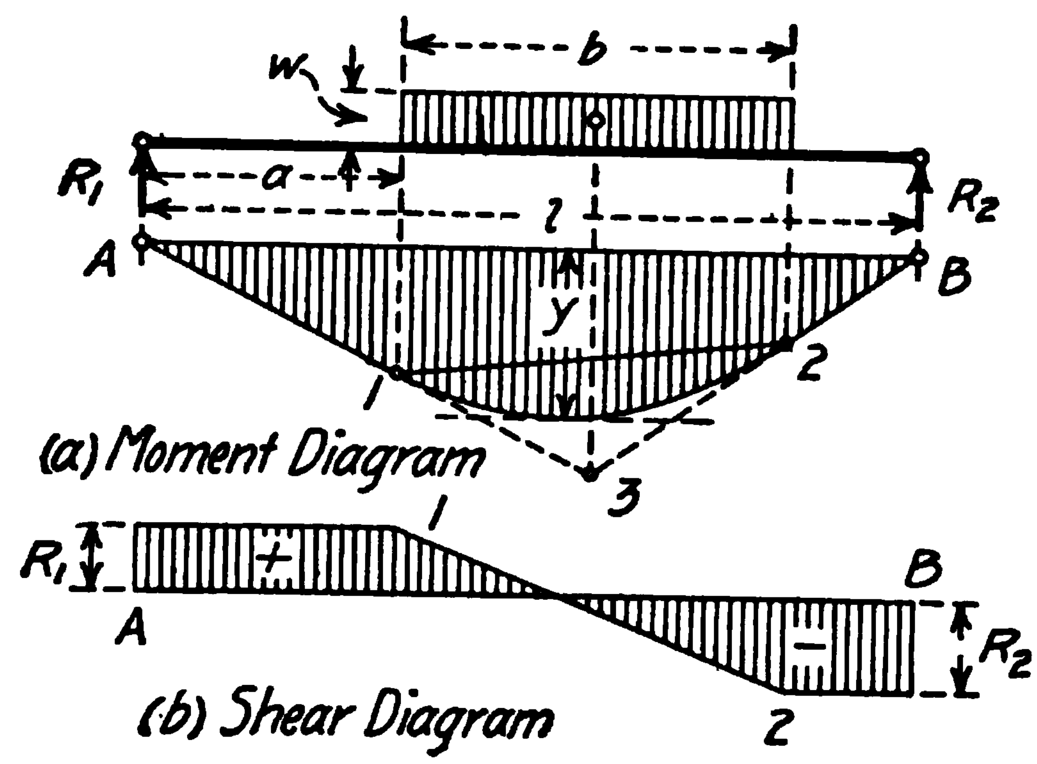


FIG. 74.

be seen that the maximum bending moment comes at the point of zero shear.

Uniform Moving Loads.—Let the beam in Fig. 75 be loaded with a uniform load of p lbs. per lineal foot, which can be moved on or off the beam.

To find the position of the moving load that will produce a maximum moment at a point a distance a from the left support, proceed as follows: Let the end of the uniform load be at a distance x from the left reaction. Then taking moments about R_2 we have

$$R_1 = \frac{(L-x)^2}{2L} p \tag{15}$$

and the moment at the point whose abscissa is a will be

$$M = R_1 \cdot a - \frac{(a-x)^2}{2} p = \frac{(L-x)^2}{2L} a \cdot p - \frac{(a-x)^2}{2} p \tag{16}$$

Differentiating (16) with respect to x, and placing the derivative of M equal to zero, we have after solving

$$x = 0 \tag{17}$$

Therefore the maximum moment at any point in a beam will occur when the beam is fully loaded.

The bending moment diagram for a beam loaded with a uniform moving load is constructed as in Fig. 73.

To find the position of the moving load for maximum shear at any point in a beam loaded with a moving uniform load, proceed as follows: The left reaction when the end of the moving load is at a distance x from the left reaction will be

$$R_1 = \frac{(L-x)^2}{2L} p \tag{15}$$

and the shear at a point at a distance a from the left reaction will be

$$S = R_1 - (a-x)p = \frac{(L-x)^2}{2L} p - (a-x)p \tag{18}$$

which is the equation of a common parabola.

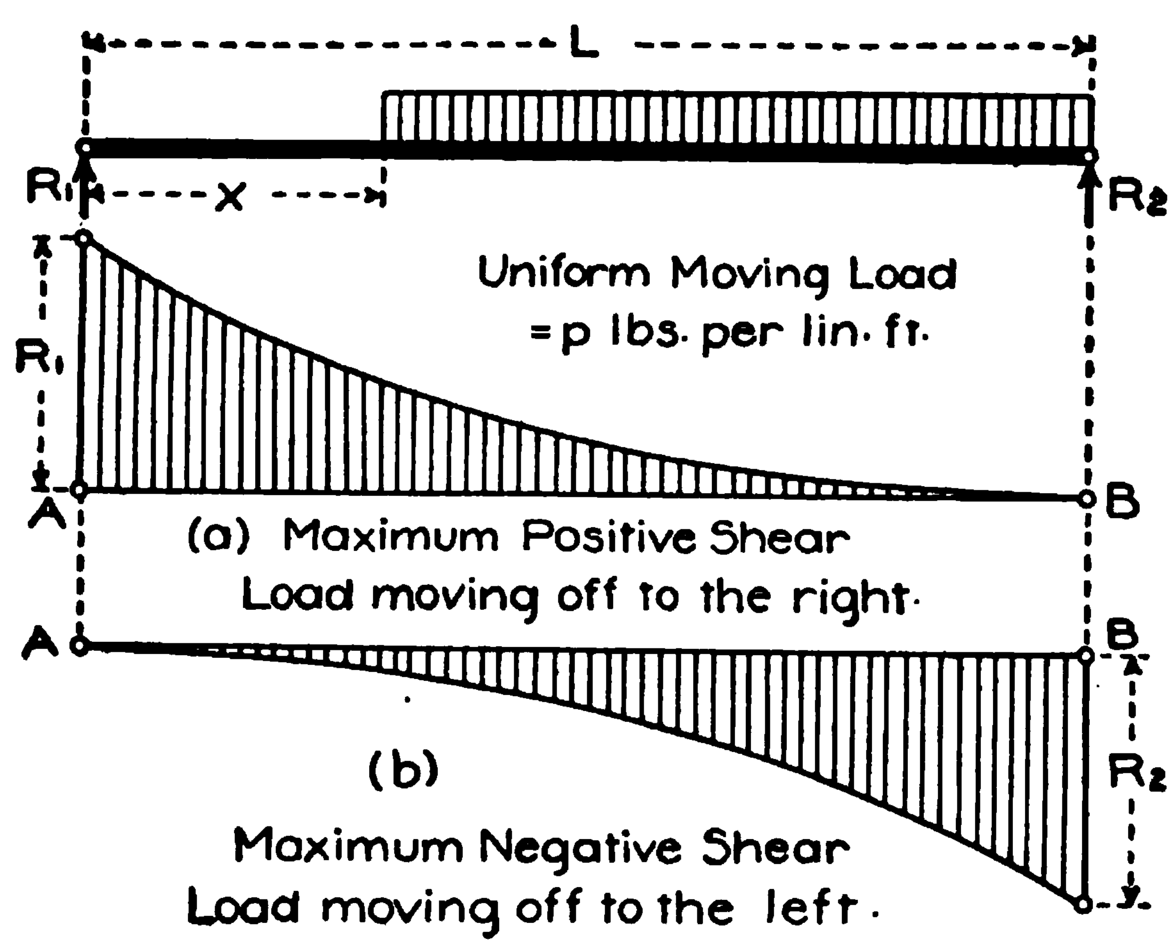


Fig. 75.

By inspection it can be seen that S will be a maximum when $x = a$. The maximum shear at any point in a beam will therefore occur at the end of the uniform moving load, the beam being fully loaded to the right of the point as in (a) Fig. 75 for maximum positive shear,

and fully loaded to the left of the point as in (b) Fig. 75 for maximum negative shear.

If the beam is assumed to be a cantilever beam fixed at A, and loaded with a stationary uniform load equal to p lbs. per lineal foot, and an equilibrium polygon be drawn with a force polygon having a pole distance equal to length of span, L, the parabola drawn through the points in the equilibrium polygon will be the maximum positive shear diagram, (a) Fig. 75. The ordinate at any point to this shear diagram will represent the maximum positive shear at the point to the same scale as the loads (for the application of this principle to bridge trusses see Fig. 87, Chapter V).

Concentrated Moving Loads. *Bending Moments.*—Let a beam be loaded with concentrated moving loads at fixed distances apart as shown in Fig. 76.

To find the position of the loads for maximum moment and the amount of the maximum moment, proceed as follows: The load P_2 will

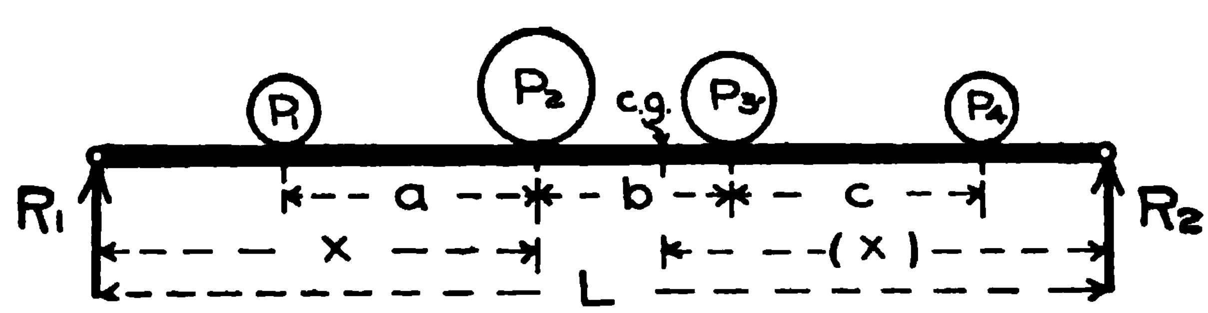


FIG. 76.

be considered first. Let x be the distance of the load P_2 from the left support, when the loads produce a maximum moment under load P_2.

Taking moments about R_2, we have

$$R_1 \cdot L = P_1(L - x + a) + P_2(L - x) + P_3(L - x - b) + P_4(L - x - b - c) = (L - x)(P_1 + P_2 + P_3 + P_4) + P_1 \cdot a - P_3 \cdot b - P_4(b + c) \quad (19)$$

and the bending moment under load P_2 will be

$$M = R_1 \cdot x - P_1 a = \frac{x(L-x)(P_1+P_2+P_3+P_4)+x[P_1 \cdot a - P_3 \cdot b - P_4(b+c)]}{L} - P_1 \cdot a \quad (20)$$

Differentiating (20) with respect to x, we have

$$\frac{dM}{dx}=\frac{(L-2x)(P_1+P_2+P_3+P_4)+P_1\cdot a-P_3\cdot b-P_4(b+c)}{L}=0 \quad (21)$$

and solving (21) for x, we have

$$x=\frac{L}{2}-\frac{P_1\cdot a-P_3\cdot b-P_4(b+c)}{2(P_1+P_2+P_3+P_4)} \quad (22)$$

Now $P_1\cdot a-P_3\cdot b-P_4(b+c)$ is the static moment of the loads about P_2, and

$$\frac{P_1\cdot a-P_3\cdot b-P_4(b+c)}{P_1+P_2+P_3+P_4}$$

= distance from P_2 to center of the gravity of all the loads.

Therefore, for a maximum moment under load P_2, it (P_2) must be as far from one end as the center of gravity of all the loads is from the other end of the beam, Fig. 76.

The above criterion holds for all the loads on the beam. The only way to find which load produces the greatest maximum is to try each one, however, it is usually possible to determine by inspection which load will produce a maximum bending moment. For example, the maximum moment in the beam in Fig. 76 will certainly come under the heavy load P_2. The above proof may be generalized without difficulty, and the criterion above shown to be of general application.

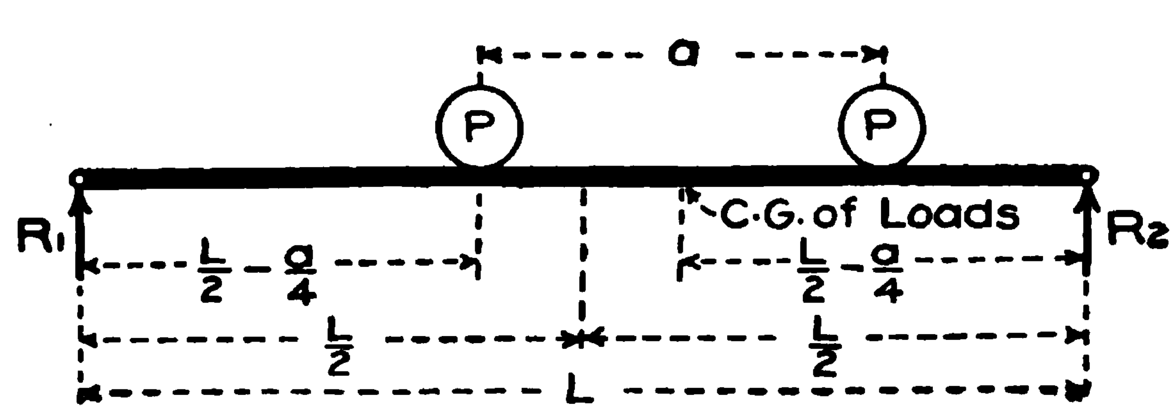


FIG. 77.

For two equal loads, $P=P$, at a fixed distance, a, apart as in the case of a traveling crane, Fig. 77, the maximum moment will occur under one of the loads, when

$$x=L/2-a/4$$

Taking moments about the right reaction, we have

$$R_1 \cdot L = P(L - a/2) \qquad (23)$$

and the maximum bending moment is

$$M = R_1(L/2 - a/4)$$

$$= \frac{P(L - a/2)^2}{2L} \qquad (24)$$

There will be a maximum moment when either of the loads satisfies the above criterion, the bending moments being equal.

By equating the maximum moment calculated as above to the moment due to a single load at the center of the beam, it will be found that the above criterion holds only, when

$$a < 0.586L \qquad (25)$$

Where two unequal moving loads are at a fixed distance apart, the greater maximum bending moment will always come under the heavier load.

Shears.—The maximum end shear at the left support, for a system of concentrated loads on a simple beam, as in Fig. 77, will occur when the left reaction, R_1, is a maximum. This will occur when one of the wheels is infinitely near the left abutment (usually said to be over the left abutment). The load which produces maximum end shear can be easily found by trial.

The maximum shear at any point in the beam will occur when one of the loads is over the point. The criterion for determining which load will cause a maximum shear at any point, x, in a beam will now be determined.

In Fig. 77, let the total load on the beam, $P_1 + P_2 + P_3 + P_4 = W$, and let x be the distance from the left support to the point at which we wish to determine the maximum shear.

When load P_1 is at the point, the shear will be equal to the left reaction, which is found by substituting $x + a$ for x in (19) to be

$$S_1 = R_1 = \frac{(L-x-a)W + P_1 \cdot a - P_3 \cdot b - P_4(b+c)}{L}$$

and when P_2 is at the point, the shear will be

$$S_2 = \frac{(L-x)W + P_1 \cdot a - P_3 \cdot b - P_4(b+c)}{L} - P_1$$

Subtracting S_2 from S_1, we have

$$S_1 - S_2 = \frac{P_1 \cdot L - W \cdot a}{L}$$

Now S_1 will be greater than S_2 if $P_1 \cdot L$ is greater than $W \cdot a$, or if

$$P_1/a > W/L$$

The criterion for maximum shear at any point, therefore, is as follows:

The maximum positive shear in any section of a beam occurs when the foremost load is at the section, provided W/L is not greater than P_1/a. If W/L is greater than P_1/a, the greatest shear will occur when some succeeding load is at the point.

Having determined the position of the moving loads for maximum moment and maximum shear, the amount of the moment and shear can be obtained as in the case of beams loaded with stationary loads.

DESIGN OF BEAMS.—Having calculated the maximum bending moments and shears in the beam the stresses are calculated as follows:

Shearing Stresses.—The shear is assumed as uniformly distributed over the cross-section of the beam, and the shearing stress will be equal to the shear S_1 divided by the area of the beam. The actual shearing stress in the beam must be less than the allowable shearing stress.

Tensile and Compression Stresses.—A simple beam carried on two end supports will have its upper fibers in compression and its lower fibers in tension, there being no stress on the neutral axis of the beam.

The stress due to bending moment will be given by the formula

$$S = M \cdot c/I \tag{26}$$

where S is the unit stress on the extreme fiber, being tension on the convex side and compression on the concave side of the beam, M is the bending moment of the forces on one side of the given section, c is the distance in inches from the neutral axis of the beam to the extreme fiber considered, and I is the moment of inertia of the cross-section of the beam in inches to the fourth power.

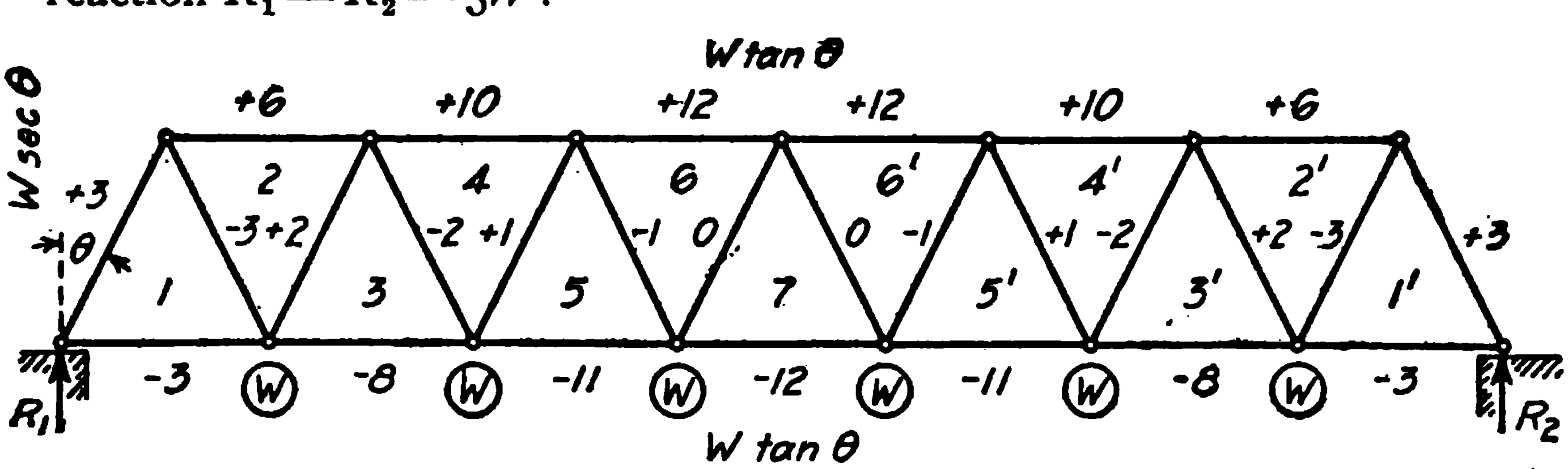
The allowable unit stresses for shear, tension and compression are given in Chapter XIII and in Appendix I.

CHAPTER V.

Stresses in Highway Bridge Trusses.

LOADS.—The loads on highway bridges are commonly specified as a certain number of pounds per square foot of floor surface, or per lineal foot of truss or bridge. The live load is assumed as applied at the panel points of the loaded chord, while the dead load may be assumed as all applied on the loaded chord, or assumed as partly applied on the loaded chord and partly on the unloaded chord (usually two-thirds on the loaded chord and one-third on the unloaded chord). In this discussion the deal load will be assumed as applied at the panel points in the loaded chord. Equal panel lengths and joint loads will also be assumed. For extracts from standard specifications for dead loads of highway bridges, see Chapter II.

Algebraic Resolution.*—Let the Warren truss, in Fig. 78, have dead loads applied at the joints of the lower chord as shown. From the fundamental equations for equilibrium for rotation and translation, reaction $R_1 = R_2 = 3W$.



Dead Load Coefficients

Fig. 78.

The stresses in the members are calculated as follows: Resolving at the left reaction, stress in 1–$x = + 3W \cdot \sec \theta$, and stress in 1–$y =$

* Also called "Method of Sections."

$-3W\cdot\tan\theta$. Resolving at first joint in upper chord, stress in 1–2 $= -3W\cdot\sec\theta$, and stress in 2–$x = +6W\cdot\tan\theta$. Resolving at second joint in lower chord, stress 2–3 $= +2W\cdot\sec\theta$, and stress 3–$y = -8W\cdot\tan\theta$. And in like manner the stresses in the remaining members are found as shown. The coefficients shown in Fig. 78 for the chords are to be multiplied by $W\cdot\tan\theta$; while those for the webs are to be multiplied by $W\cdot\sec\theta$.

It will be seen that the coefficients for the web stresses are equal to the shears in the respective panels. Having found the shears in the different panels of the truss, the remaining coefficients may be found by resolution. Pass a section through any panel and the algebraic sum of the coefficients will be equal to zero. Therefore, if two coefficients are known, the third will be equal to the algebraic sum of the two, with sign changed.

Beginning with coefficient of member 1–y, which is known and equals -3;

$$\begin{aligned}
&\text{coefficient of } 2\text{–}x = -(-3-3) = +6;\\
&\text{coefficient of } 3\text{–}y = -(+6+2) = -8;\\
&\text{coefficient of } 4\text{–}x = -(-8-2) = +10;\\
&\text{coefficient of } 5\text{–}y = -(+10+1) = -11;\\
&\text{coefficient of } 6\text{–}x = -(-11-1) = +12;\\
&\text{coefficient of } 7\text{–}y = -(+12+0) = -12.
\end{aligned}$$

Loading for Maximum Stresses.—The effect of different positions of the loads on a Warren truss will now be investigated.

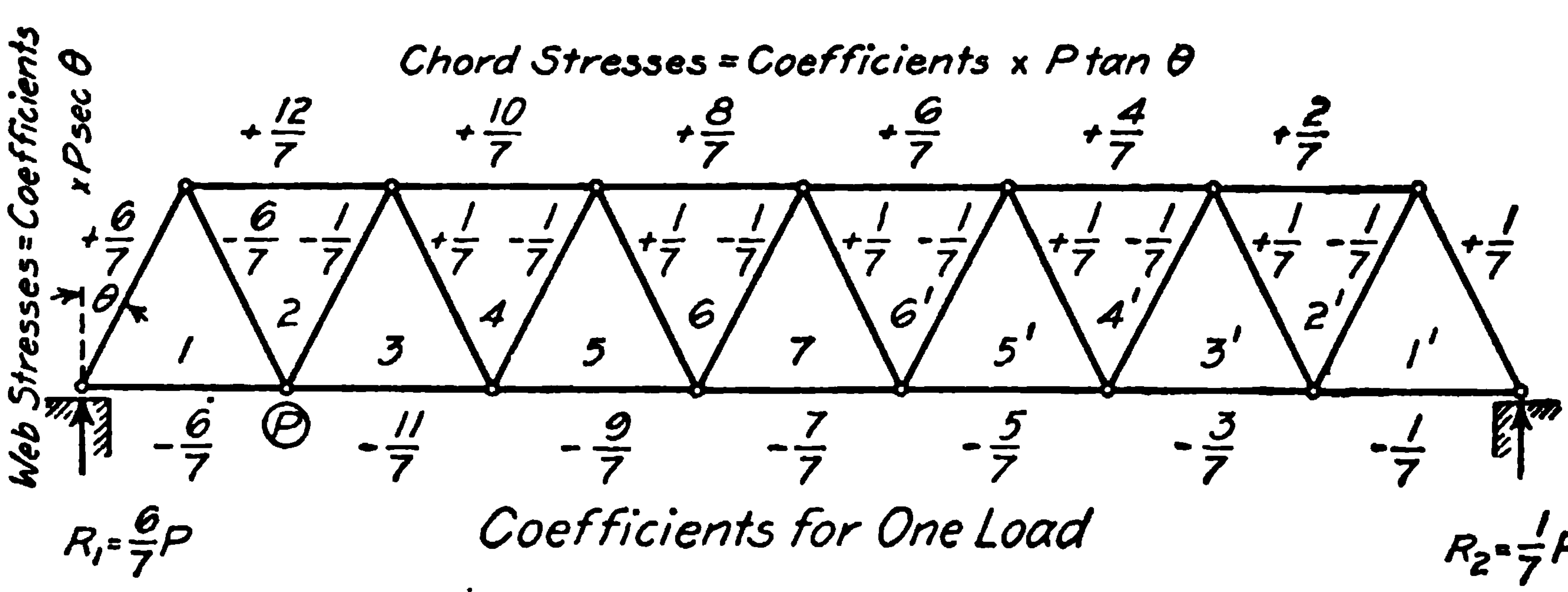
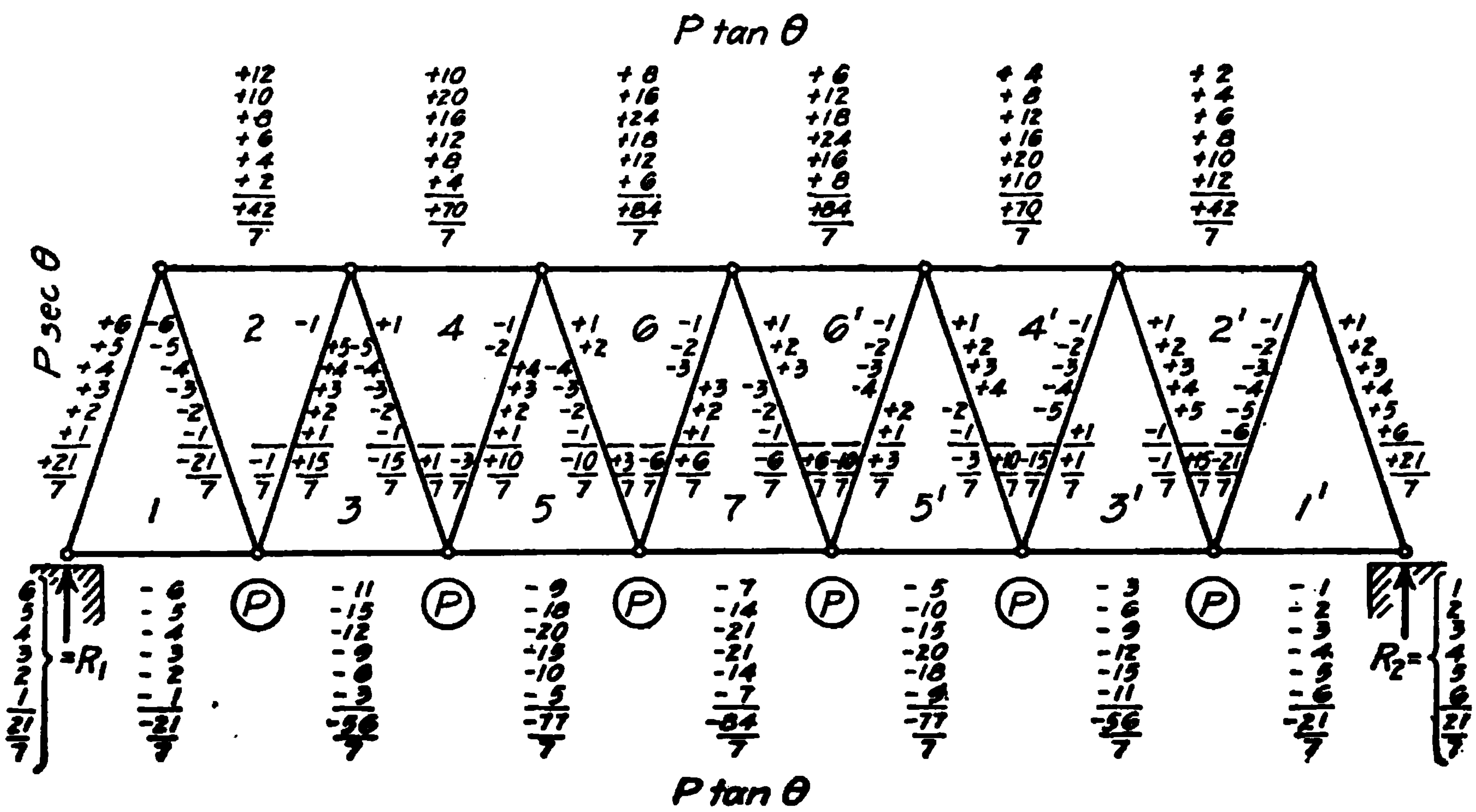


FIG. 79.

Let the truss in Fig. 79 be loaded with a single load P as shown. The left reaction, $R_1 = 6/7 P$, and the right reaction, $R_2 = P/7$. The stress in 1–y $= -6/7 P \cdot \tan\theta$, and stress in 1–$x$ $= +6/7 P \cdot \sec\theta$. The stress in 1–2 $= -6/7 P \cdot \sec\theta$, and stress in 2–3 $= -1/7 P \cdot \sec\theta$, etc. The remaining coefficients are found as in the case of dead loads by adding coefficients algebraically and changing the sign of the result.

In Fig. 80 the coefficients for a load applied at each joint in turn are shown for the different members; the coefficients for the load on left being given in the top line.



Maximum and Minimum Coefficients

FIG. 80.

The following conclusions may be drawn from Fig. 80:

1. All loads produce compressive stresses in the top chord and tensile stresses in the bottom chord.

2. All the loads on one side of a panel produce the same kind of stress in the web members that are inclined in the same direction on that side.

3. For maximum stresses in the chords, therefore, the truss should be fully loaded.

4. For maximum stresses in the web members the longer segment into which the panel divides the truss should be fully loaded; while

for minimum stresses in the web members the shorter segment of the truss should be fully loaded.

The conditions for maximum loading of a truss with equal joint loads are therefore seen to be essentially the same as the maximum loading of a beam with a uniform live load.

For a discussion of the conditions of loading for maximum and minimum stresses in trusses by means of Influence Diagrams, see Chapter VI.

Stresses in a Warren Truss.—The coefficients for the maximum and minimum stresses in a Warren truss, due to live load are shown in Fig. 81.

These coefficients are seen to be the algebraic sum of the coefficients for the individual loads given in Fig. 80. The live load chord

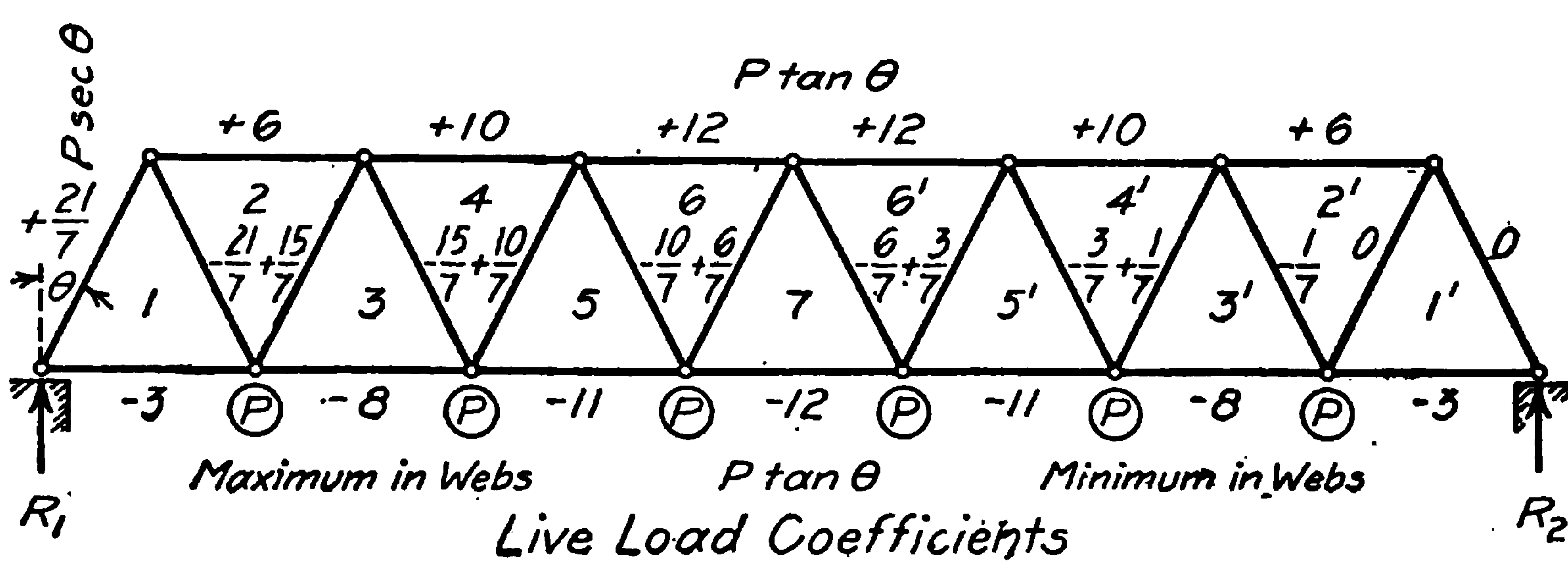


FIG. 81.

coefficients are the same as for dead load, and if found directly are found in the same manner.

The maximum web coefficients may be found directly by taking off one load at a time, beginning at the left. The left reaction, which may be found by algebraic moments, will in each case be the coefficient of the maximum stress in the panel to the left of the first load. A rule for finding the coefficient of left reaction for any loading is as follows: *Multiply the number of loads on the truss by one-half the number of loads plus unity, and divide the product by the number of panels in the truss, the result will be the coefficient of the left reaction.*

If the second differences of the maximum coefficients in the web

members are calculated, they will be found to be constant, which shows that the coefficients are equal to the ordinates of a parabola.

Coefficients,	21		15		10		6		3		1
1st differences,		6		5		4		3		2	
2nd differences,			1		1		1		1		

SECOND DIFFERENCES OF NUMERATORS OF WEB COEFFICIENTS.

This relation gives an easy method for checking up the maximum web coefficients, since the numerators of the coefficients are always the same beginning with unity in the first panel on the right and progressing in order 1, 3, 6, 10, etc.; the denominators always being the number of panels in the truss.

It will also be found that the second differences of the upper or lower chord coefficients are constant, showing that the chord stresses are proportional to the ordinates to a parabola.

It should be noted that in the Warren truss the web members meeting on the unloaded chord always have stresses equal in amount, but opposite in sign.

The web member 6–7 has a zero dead load stress, and a complete reversal due to live load, making it necessary to design the member to take both tension and compression.

For the calculation of the maximum and minimum stresses in a Warren truss, see Problem 10, Chapter IX.

Stresses in a Pratt Truss.—In the Pratt truss the diagonal members are tension members, and counters (see dotted members in (c) Fig. 82) must be supplied where there is a reversal of stress. The coefficients for the dead and live load stresses in the Pratt truss, shown in (a) and (b) Fig. 82, are found in the same manner as for a Warren truss. The member U_1L_1 acts as a hanger and carries only the load at its lower end. The stresses in the chords are found by multiplying the coefficients by $W \cdot \tan\theta$, and in the inclined webs by multiplying the coefficients by $W \cdot \sec\theta$. The stresses in the posts are equal to the vertical components of the stresses in the inclined web members meeting them on the unloaded chord.

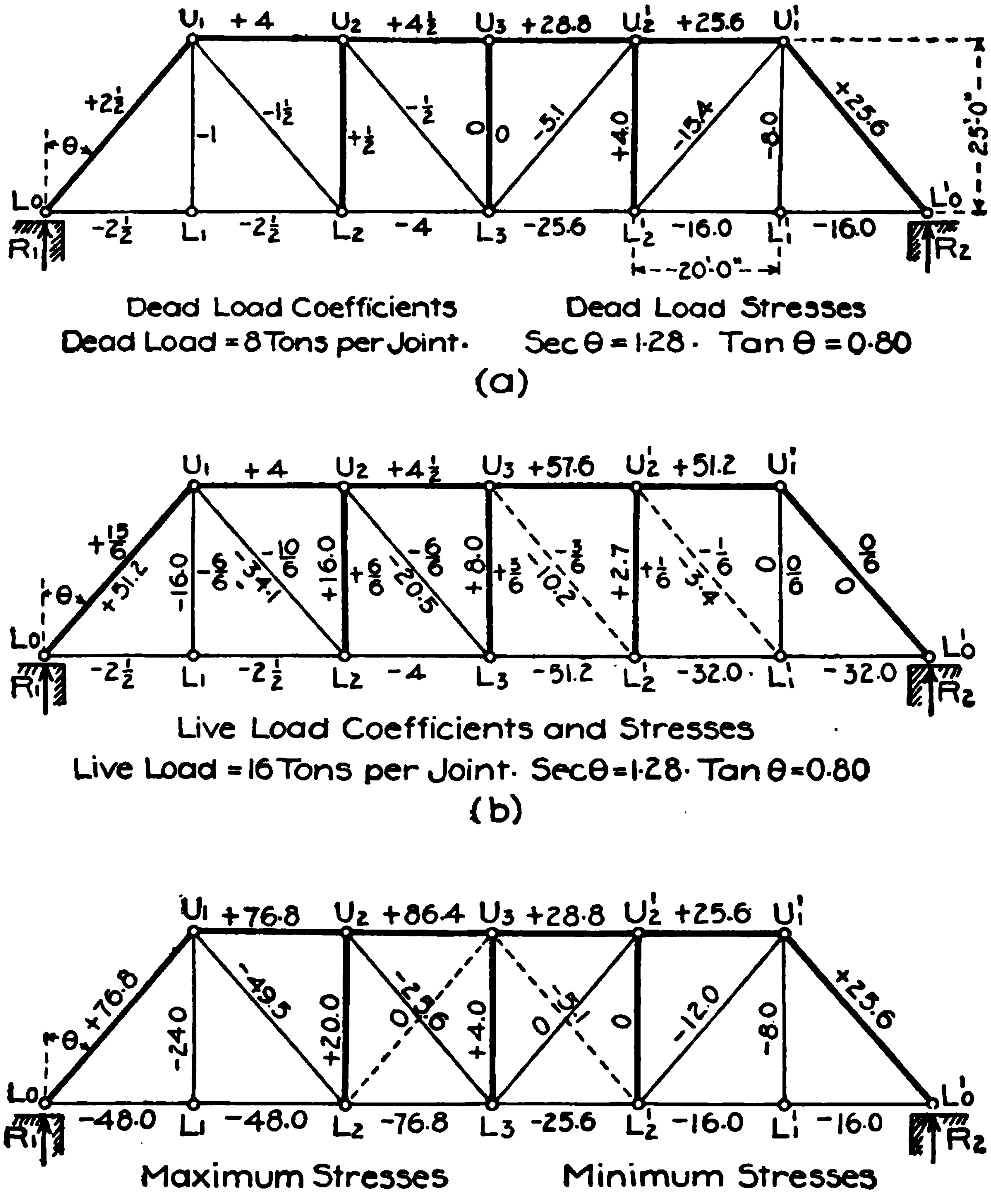


FIG. 82.

The maximum chord stresses shown on the left of (c), are equal to the sum of the live and dead load chord stresses. The minimum chord stresses shown on the right of (c), are equal to the dead load chord stresses.

The maximum and minimum web stresses are found by adding, algebraically, the stresses in the members due to dead and live loads.

Since the diagonal web members in a Pratt truss can take tension

only, counters must be supplied as U_3L_2' in panel $L_2'L_3$. The tensile stress in a counter in a panel of a Pratt truss is always equal to the compressive stress that would occur in the main diagonal web member in the panel, if it were possible for it to take compression. Care must always be used to calculate the corresponding stresses in the vertical posts.

For the calculation of the maximum and minimum stresses in a Pratt truss, see Problem 11, Chapter IX.

Method of Shear Increments.—The loads on a beam or truss first produce shears, which in turn produce bending stresses in the chords. In (a) Fig. 82 it will be seen that member U_2L_3 carries the shear in the panel of $\frac{1}{2}W$, which produces a stress of $-\frac{1}{2}W\cdot\sec\theta$ in the member. The difference in the stresses in U_1U_2 and U_2U_3 is seen to be the horizontal component of the stress in U_2L_3, or the shear increment in the panel. The shear increment may be calculated as follows: The shear in the panel L_2L_3 is $W/2$ and may be assumed to act a differential to the right of joint L_2. Now take moments about L_3, and pass a section cutting U_2U_3, U_2L_3 and L_2L_3 just to the right of L_2, and cutting away the truss to the left. Now the shear, S, represents the resultant of the vertical forces to the left of the panel. Then for equilibrium the stress in U_2U_3 will be equal to the stress in U_1U_2 found by taking moments about joint L_2, plus the shear increment $I = (S \times l)/d$, $= \frac{1}{2}W\cdot\tan\theta$, where $l =$ panel length and $d =$ depth of truss.

GRAPHIC RESOLUTION.—The stresses in a Warren truss due to dead loads are calculated by graphc resolution in Problem 1, Chapter IX. The solution is the same as for the truss in Fig. 58. The loads, beginning with the first load on the left, are laid off from the bottom upwards. The analysis of the solution is shown on the stress diagram and truss, and needs no explanation.

From the stresses in the members it is seen (a) that web members meeting on the unloaded chord have stresses equal in amount but opposite in sign, and (b) that the lower chord stresses are the arithmetical means of the upper chord stresses on each side.

The live load chord stresses may be obtained from the dead load

stress diagram, by changing the scale, or by multiplying the dead load stresses by a constant.

The live load web stresses may be obtained by calculating the left reactions for the loading that gives a maximum shear in the panel (no loads occurring between the panel and the left reaction), and then constructing the stress diagram up to the member whose stress is required. In a truss with parallel chords it is only necessary to calculate the stress in the first web member for any given reaction, since the shear is constant between the left reaction and the panel in question.

The live load web stresses may all be obtained from a single diagram as follows: With an assumed left reaction of, say, 100,000 lbs. construct a stress diagram on the assumption that the truss is a cantilever fixed at the right abutment, and that there are no loads on the truss. Then the maximum stress in any web member will be equal to the stress scaled from the diagram, divided by 100,000, multiplied by the left reaction that produces the maximum stress. This method is a very convenient one for finding the stresses in a truss with inclined chords. For an example, see Problem 19, Chapter IX.

In calculating the maximum and minimum stresses in a bridge truss by graphic resolution the labor in constructing the stress diagram may be reduced by replacing the truss to the left of the panel by a triangle as in (a) or (b) in Fig. 84. In (a) the correct stresses will be given in U_3L_3' or $U_3'L_3$, but the correct stress will not be given in U_3L_3.

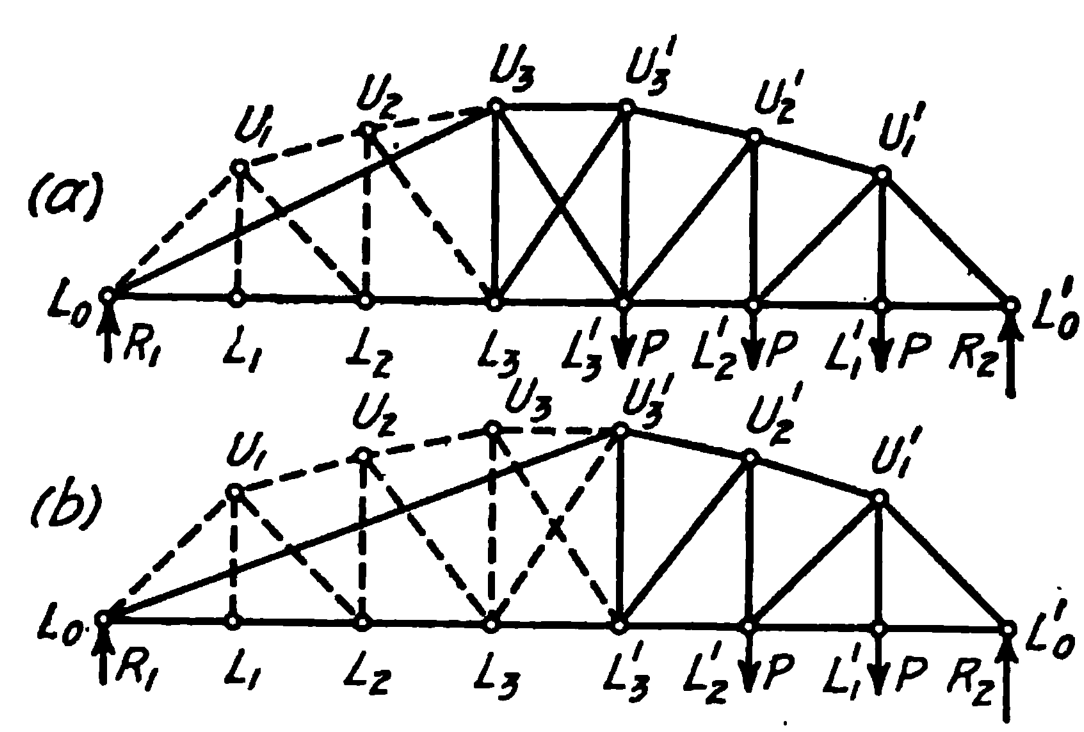


Fig. 84.

ALGEBRAIC MOMENTS.—The dead and live load stresses in a truss with inclined chords are calculated by algebraic moments in Fig. 85. The conditions for maximum loading are the same in this truss as in a truss with parallel chords, and are as follows: Maximum chord stresses occur when all loads are on; minimum chord stresses occur when no live load is on; maximum web stresses in main members occur when the longer segment of the truss is loaded; and minimum stresses in main members and maximum stresses in counters occur when the shorter segment of the truss is loaded. For a proof of this criterion, see Fig. 93, Chapter VI. An apparent exception to the latter rule occurs in post U_2L_2, which has a maximum tensile stress when the truss is fully loaded with dead and live loads.

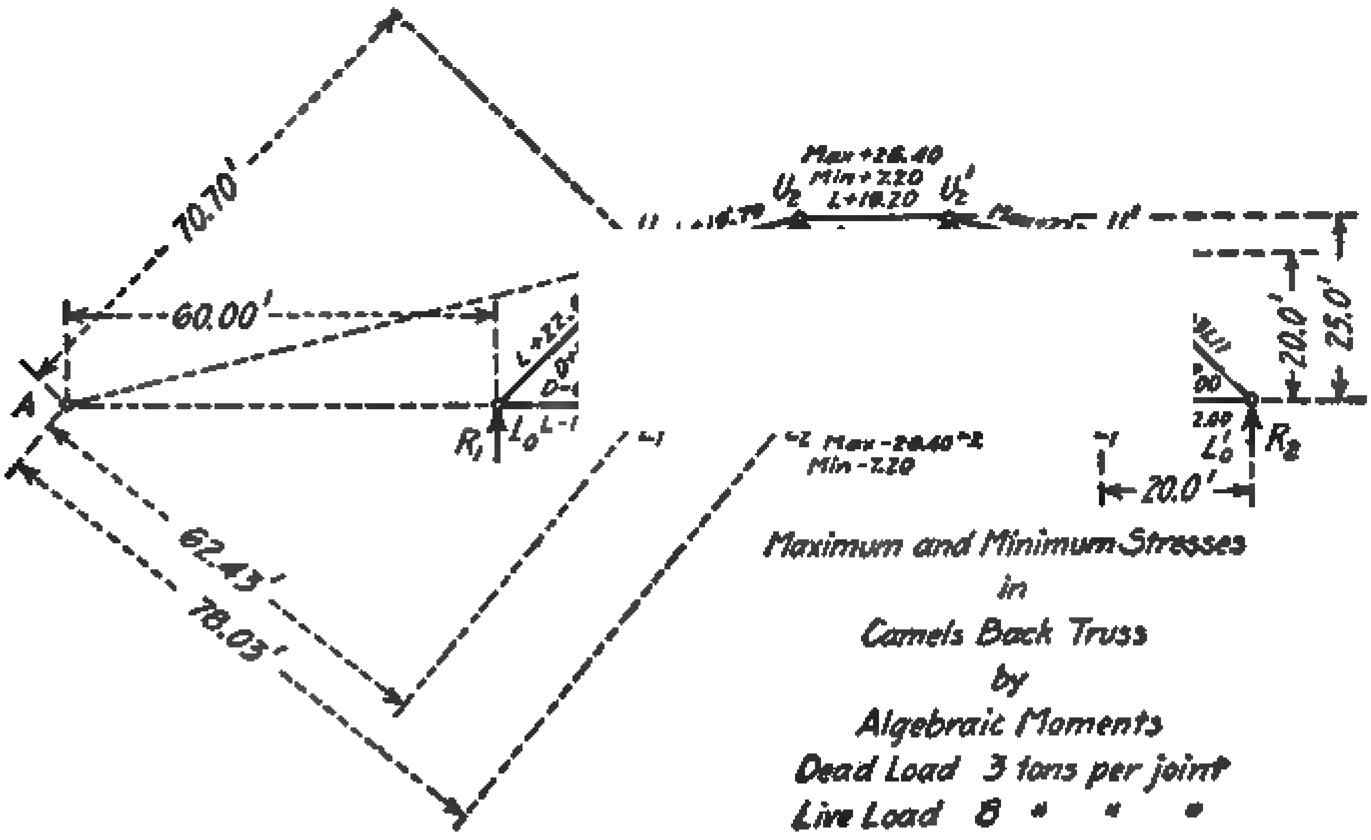


Fig. 85.

To calculate the stress in member U_1L_2, take moments about point A, the intersection of the upper and lower chords produced and pass a section cutting U_1U_2, U_1L_2 and L_1L_2, and cutting away the truss to the right. The dead load stress is then given by the equation

$$U_1L_2 \times 70.7 + R_1 \times 60 - W \times 80 = 0$$

$$U_1L_2 \times 70.7 = -6 \times 60 + 3 \times 80 = -120 \text{ foot-tons, and}$$

$$U_1L_2 = -1.70 \text{ tons}$$

The maximum live load stress occurs when all loads are on except L_1, and

$$U_1L_2 \times 70.7 + R_1 \times 60 = 0$$
$$U_1L_2 \times 70.7 = -\tfrac{6}{5}P \times 60 = -576 \text{ foot-tons, and}$$
$$U_1L_2 = -8.14 \text{ tons}$$

The maximum live load stress in counter U_2L_1 occurs with a load at L_1, and is given by the equation

$$-U_2L_1 \times 62.43 + R_1 \times 60 - P \times 80 = 0$$
$$U_2L_1 \times 62.43 = \tfrac{4}{5}P \times 60 - 8 \times 80 = 256 \text{ foot-tons, and}$$
$$U_2L_1 = -4.10 \text{ tons}$$

The dead load stress in counter U_2L_1 when main member U_1L_2 is not acting will be

$$U_2L_1 \times 62.43 = +120 \text{ foot-tons, and}$$
$$U_2L_1 = +1.92 \text{ tons}$$

The maximum stress in U_1L_2 is therefore $-1.70 - 8.14 = -9.84$ tons, and the minimum stress is zero. The maximum stress in counter U_2L_1 is $+1.92 - 4.10 = -2.18$ tons, and the minimum stress is zero.

To calculate the stress in member U_1U_2, take the center of moments at L_2, and pass a section cutting U_1U_2, U_2L_2 and L_2L_2', and cutting away the truss to the right. The dead load stress is then given by the equation

$$U_1U_2 \times 24.25 - R_1 \times 40 + W \times 20 = 0$$
$$U_1U_2 = +7.42 \text{ tons}$$

In like manner the live load stress in $U_1U_2 = +19.79$ tons.

The stresses in the remaining members may be found in the same manner. To obtain stress in upper chord U_2U_2', take moments about L_2 as a center; to obtain stress in lower chord L_0L_1 take moments about U_1 as a center. The dead load and maximum live load tensile stress in post U_2L_2 is equal to the vertical component of the dead and live loads, respectively, in upper chord U_1U_2. The stresses in L_0U_1, L_0L_1, L_2L_2', U_2U_2' and U_2L_2' are most easily found by algebraic resolution.

For additional problems, see Chapter IX.

GRAPHIC MOMENTS.—The dead load stresses in the chords of a Warren truss are calculated by graphic moments in Fig. 86.

Bending Moment Polygon.—The upper chord stresses are given by the ordinates to the bending moment parabola direct, while the lower chord stresses are arithmetical means of the upper chord stresses on each side, and are given by the ordinates to the chords of the parabola as shown in Fig. 86.

The parabola is constructed as follows: The mid-ordinate, 4–j, is made equal to the bending moment at the center of the truss divided by the depth; in this case the mid-ordinate is the stress in 6–x; if the

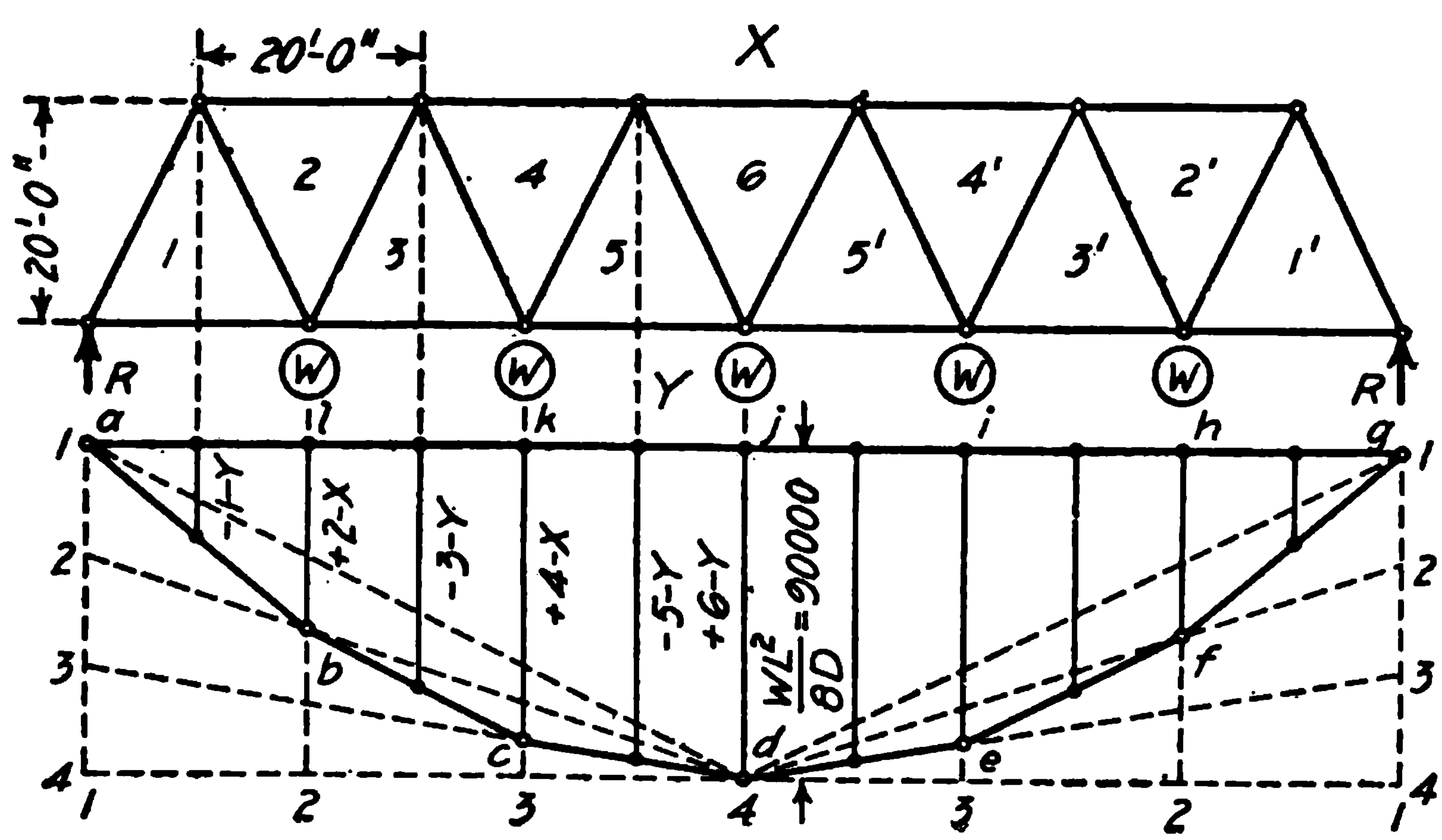


FIG. 86.

number of panels in the truss were odd, the mid-ordinate would not be equal to any chord stress. The parabola is then constructed as shown in Fig. 86. The live load chord stresses may be found from Fig. 86 by changing the scale, or by multiplying the dead load chord stresses by a constant.

Shear Polygon.—In Chapter IV it was shown that the maximum shear in a beam at any point could be represented by the ordinate to a parabola at the point. The same principle holds for a symmetrical bridge truss with equal panels and loaded with equal joint loads, as will now be proved.

In Fig. 87 assume that the simple Warren truss is fixed at the left end as shown, and that the right reaction R_2 is not acting. Then with all joints fully loaded with a live load P, construct a force polygon as

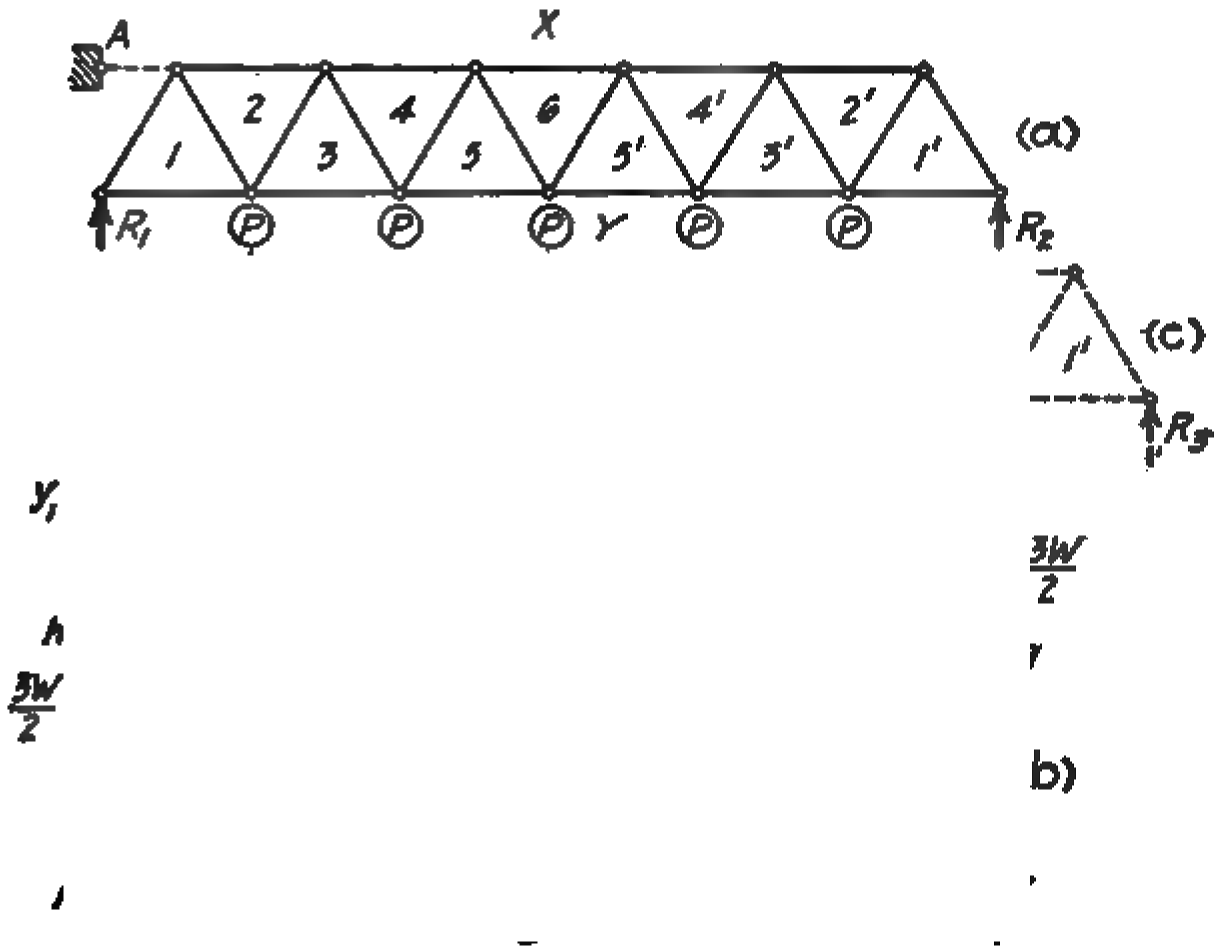


FIG. 87.

shown, with pole O and pole distance, H = span L, and beginning at point a in the load line of the force polygon, construct the equilibrium polygon a–g–h for the cantilever truss.

Now the bending moment at the left support will be equal to ordinate y_1 multiplied by the pole distance H. But the truss is a simple truss and the moment of the right reaction will be equal to the moment at the left abutment, and

$$y_1 \cdot H = R_2 \cdot L$$

and since $H = L$

$$y_1 \cdot L = R_2 \cdot L$$

and

$$y_1 = R_2$$

Now, with the loads remaining stationary, move the truss one panel to the right as shown by the dotted truss. With the same force polygon draw a new equilibrium polygon as above. This equilibrium polygon will be identical with a part of the first equilibrium polygon as shown. As above, the bending moment at left reaction is $y_8 \cdot H = y_8 \cdot L = R_8 \cdot L$, and $y_8 = R_8$. In like manner y_6 can be shown to be the right reaction with three loads on, etc. Since the bridge is symmetrical with reference to the center line, the ordinates to the shear polygon in Fig. 87 are equal to the maximum shears in the panel to the right of the ordinate as the load moves off the bridge to the right.

For a method of drawing the shear parabola direct, without the use of the force and equilibrium polygons, see Problem 18, Chapter IX.

CHAPTER VI.

STRESSES IN RAILWAY BRIDGE TRUSSES.

LOADS.—The dead load of a railway bridge is assumed to act at the joints the same as in a highway bridge. The dead joint loads are commonly assumed to act on the loaded chord, but may be assumed as divided between the panel points of the two chords, one-third and two-thirds of the dead loads usually being assumed as acting at the panel points of the unloaded and the loaded chords, respectively.

The live load on a railway bridge consists of wheel loads, the weights and spacing of the wheels depending upon the type of the rolling stock used. The locomotives and cars differ so much that it would be difficult if not impossible to design bridges on a railway system for the actual conditions, and conventional systems of loading, which approximate the actual conditions are assumed. The conventional systems for calculating the live load stresses in railway bridges that have been most favorably received are: (1) Cooper's Conventional System of Wheel Concentrations; (2) the use of an Equivalent Uniform Load; and (3) the use of a uniform load and one or two wheel concentrations. In addition to these some railroads specify special engine loadings. The first and second methods will be discussed in this chapter.

Cooper's Conventional System of Wheel Concentrations.—In Cooper's loadings two consolidation locomotives are followed by a uniformly distributed train load. The typical loading for Cooper's Class E 40 is shown in Fig. 47. The loads on the drivers in thousands of pounds and the uniform train load in hundreds of pounds are the same as the class number. The wheel spacings are the same for all classes. The stresses for Cooper's loadings calculated for one class may be used to obtain the stresses due to any other class loading. For example, the stresses in any truss due to Cooper's Class E 50 are equal

to $\frac{5}{4}$ of the stresses in the same truss due to Class E 40 loading. The E 40 and the E 50 loadings are those most used for steam railways in the United States. In bridges designed for Class E 40 loading and under the floor system must in addition be designed for two moving loads of 100,000 lbs. each, spaced 6′ 0″ apart on each track. The corresponding loads for Class E 50 are 120,000 lbs. with the same spacing. The American Railway Engineering and Maintenance of Way Association has adopted Cooper's loadings, except that the special loads are spaced 7′ 0″.

Equivalent Uniform Load System.—The equivalent uniform load for calculating the stresses in trusses and the bending moments in beams, is the uniform load that will produce the same bending moment at the quarter points of the truss or beam as the maximum bending moment produced by the wheel concentrations. The equivalent uniform loadings for different spans for Cooper's E 40 loading are given in Fig. 48. In calculating the stresses in the truss members select the equivalent load for the given span, and calculate the chord and web stresses by the use of equal joint loads, as for highway bridges. In designing the stringers for bending moment take a loading for a span equal to one panel length, and for the maximum floorbeam reaction take a loading for a span equal to two panel lengths. It is necessary to calculate the maximum end shears and the shears at intermediate points by wheel concentrations, or to use equivalent uniform loads calculated for wheel concentrations. The equivalent uniform loads for moment, M, shear, S, and floorbeam reaction, R, for Class E 40 are given in Table X.

The equivalent uniform load method has been advocated very strongly by Mr. J. A. L. Waddell who has described its use in detail in his "De Pontibus."

Live load stresses calculated by the method of equivalent uniform loads are too small for the chords and webs between the ends of the truss and the quarter points, and are too large between the quarter points. The stresses obtained for the counters are too large. The live load stresses calculated by the method of equivalent uniform loads are

TABLE X.

MAXIMUM MOMENTS M, END SHEARS S, AND FLOORBEAM REACTIONS R, PER TRACK, FOR COOPER'S LOADING E 40, FOR GIRDER BRIDGES.

Span L. Ft.	Max. Mom. M. Ft.-lbs.	Max. End Shear S. Lbs.	Max. Floor Reac. R. Lbs.	Equivalent Uniform Load.		
				M. Lbs.	S. Lbs.	R. Lbs.
10	112,500	60,000	80,000	9,000	12,000	8,000
11	131,400	65,500	87,300	8,690	11,910	7,940
12	160,000	70,000	93,300	8,890	11,670	7,770
13	190,000	73,800	98,500	9,000	11,350	7,580
14	220,000	77,200	104,300	8,980	11,030	7,450
15	250,000	80,000	109,300	8,890	10,670	7,290
16	280,000	85,000	113,700	8,750	10,620	7,110
17	310,000	89,500	117,600	8,580	10,530	6,920
18	340,000	93,400	121,300	8,400	10,380	6,740
19	373,200	96,800	125,800	8,270	10,190	6,620
20	412,500	100,000	131,100	8,250	10,000	6,560
21	452,000	102,800	136,000	8,200	9,790	6,480
22	491,400	105,500	140,300	8,120	9,590	6,380
23	530,800	107,900	144,300	8,030	9,380	6,270
24	570,400	110,800	148,000	7,920	9,230	6,170
25	610,000	113,600	151,300	7,810	9,090	6,050
26	649,600	116,100	155,400	7,690	8,930	5,970
27	689,200	118,500	160,100	7,560	8,780	5,930
28	731,000	120,800	164,600	7,460	8,630	5,875
29	775,800	123,100	168,700	7,370	8,490	5,820
30	821,000	126,100	172,500	7,300	8,410	5,750
31	865,700	128,800	176,900	7,210	8,310	5,710
32	910,800	131,500	182,000	7,120	8,220	5,690
33	955,600	133,900	186,700	7,020	8,110	5,660
34	1,000,700	136,100	191,100	6,920	8,010	5,620
35	1,046,000	138,400	195,200	6,840	7,910	5,570
36	1,097,000	141,100		6,770	7,840	
37	1,148,500	143,800		6,710	7,770	
38	1,200,000	146,200		6,650	7,700	
39	1,253,500	148,600		6,590	7,620	
40	1,311,000	150,800		6,560	7,540	
42	1,427,000	156,200		6,480	7,460	
44	1,543,000	161,100		6,370	7,320	
46	1,659,000	165,600	Trestles	6,280	7,200	
48	1,776,000	169,600	30 and 60 feet	6,170	7,070	
50	1,902,000	174,200	spans, 238,900.	6,090	6,970	
52	2,030,000	178,500		6,010	6,870	
54	2,162,000	182,400		5,930	6,760	
56	2,304,000	186,000		5,880	6,640	
58	2,446,000	190,800	40 and 60 feet	5,820	6,580	
60	2,599,000	195,200	spans, 262,900.	5,780	6,510	
62	2,753,000	200,200		5,730	6,460	
64	2,911,000	205,200		5,690	6,410	
66	3,079,000	210,000		5,660	6,360	
68	3,247,000	215,600		5,610	6,340	

TABLE X. (*Continued.*)

Span L. Ft.	Max. Mom. M. Ft.-lbs.	Max. End. Shear S. Lbs.	Max. Floor Reac. R. Lbs.	Equivalent Uniform Load. M. Lbs.	S. Lbs.	R. Lbs.
70	3,415,000	221,000		5,580	6,310	
72	3,584,000	226,700		5,540	6,300	
74	3,758,000	232,600		5,490	6,290	
76	3,942,000	238,100		5,460	6,270	
78	4,129,000	243,400		5,430	6,240	
80	4,321,000	248,400		5,400	6,210	
82	4,513,000	253,800		5,370	6,190	
84	4,713,000	259,000		5,340	6,190	
86	4,919,000	264,200		5,320	6,150	
88	5,128,000	269,400		5,300	6,120	
90	5,341,000	274,500		5,280	6,100	
92	5,552,000	279,600		5,250	6,080	
94	5,771,000	284,700		5,230	6,060	
96	5,988,000	289,600		5,200	6,030	
98	6,213,000	295,000		5,180	6,020	
100	6,440,000	300,000		5,150	6,000	
105	7,075,000	312,200		5,150	5,950	
110	7,774,000	324,000		5,140	5,890	
115	8,490,000	335,800		5,140	5,840	
120	9,228,000	347,400		5,130	5,790	
125	9,993,000	358,800		5,120	5,740	

Note.—For all other classes, the above values to be proportional to the classes.

sufficiently accurate for all practical purposes. Even though the equivalent uniform load method is simple to apply and gives sufficiently accurate results, it appears to be losing ground.

KINDS OF STRESS.—The live loads on a railway bridge produce stresses as follows:

(1) Vertical stresses due to the live load in any position;

(2) Vibratory stresses due to the moving of the live load, generally included in the term "Impact";

(3) Horizontal static stresses due to centrifugal forces, if the train is on a curve;

(4) Longitudinal static stresses due to the momentum of the train, and the friction on the rails when the brakes are applied.

Vibratory stresses cannot be calculated with our present knowledge, but are provided for by taking a percentage of the static live load as "Impact Stress," or by using smaller working stresses. Horizontal and static stresses can be calculated.

CALCULATION OF STRESSES DUE TO WHEEL CONCENTRATIONS.—The maximum stresses in any member of a truss may be found by trial, that is, by assuming a number of positions of the live load, calculating the stress for each position, and then comparing the results. This method is long and tiresome and considerable time may be saved by the application of certain simple criteria, which will now be developed by means of influence diagrams. These criteria may also be developed by algebraic methods.

INFLUENCE DIAGRAMS.—An influence diagram (commonly called an influence line) shows the variation of the effect of a moving load or a system of loads on a beam or truss. The difference between bending moment or shear diagrams and influence diagrams is that the bending moment and the shear diagram gives the moment and shear, respectively, at any point for a fixed system of loads, while an influence diagram gives the moment or shear, etc., at a fixed point for a moving system of loads. Influence diagrams are used principally for finding the position of moving loads that will produce maximum shears, moments, reactions, or stresses, although they may be used for calculating the quantities themselves. For convenience, where a number of loads are considered, the influence diagrams are drawn for a single unit load. The unit influence diagram may then be used for any load by multiplying by the given load. The unit influence diagram will be referred to in the following discussion.

Maximum Moment in a Truss or Beam.—Let P_1, in Fig. 88, represent the summation of the moving loads to the left of the panel point $2'$, and P_2 be the summation of the moving loads to the right.

The influence diagram for the point $2'$ is constructed by calculating the bending moment at $2'$ due to a unit load $= a(L-a)/L =$ ordinate 2–4, and drawing lines 1–2 and 2–3. The equation of the line 1–2 is $y = x(L-a)/L$, and the equation of the line 2–3 is

$$y = a(L-x)/L.$$

Now when $x = a$ the two lines have a common ordinate which is equal to $a(L-a)/L$. Also when $x = L$ the ordinate to 1–2 $= L-a$;

while when $x=0$, the ordinate to 2–3 is a, as is seen in Fig. 88. This relation gives an easy method of constructing an influence diagram for moments for any point in a beam or truss.

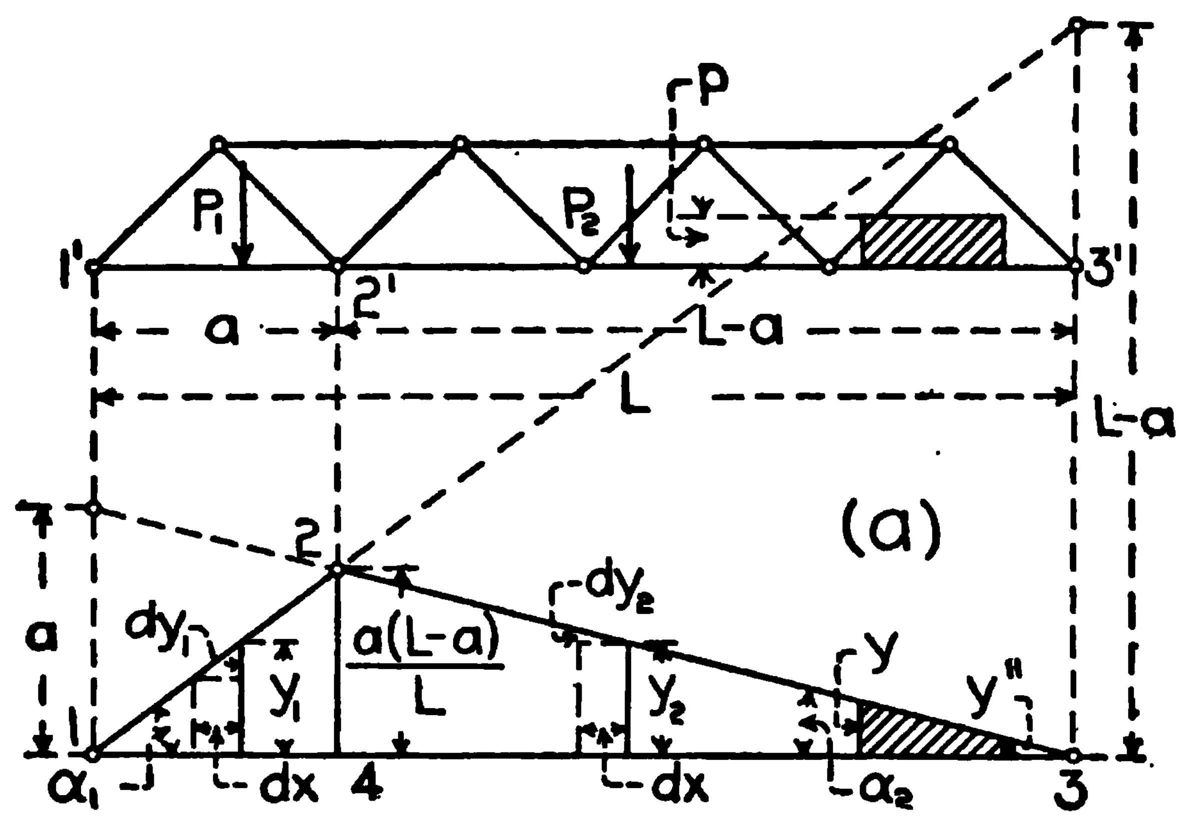


FIG. 88. INFLUENCE DIAGRAM FOR MOMENTS.

Now in Fig. 88 the bending moment at $2'$ due to the loads P_1 and P_2, is

$$M=P_1\cdot y_1+P_2\cdot y_2 \tag{27}$$

Now move the loads P_1 and P_2 a short distance to the left, the distance being assumed so small that the distribution of the loads will not be changed, and

$$M+dM=P_1(y_1-dy_1)+P_2(y_2+dy_2) \tag{28}$$

Subtracting (27) from (28) and placing $dM=0$, we have

$$dM=-P_1\cdot dy_1+P_2\cdot dy_2=0 \tag{29}$$

But $dy_1=dx\cdot\tan\alpha_1=dx(L-a)/L.$, and $dy_2=dx\cdot\tan\alpha_2=dx\cdot a/L$, and

$$dM=-P_1(L-a)\,dx/L+P_2\cdot a\cdot dx/L=0, \text{ from which}$$
$$P_1\cdot a-P_1\cdot L+P_2\cdot a=0, \text{ and}$$
$$(P_1+P_2)a=P_1L$$

Solving, we have

$$P_1/a=(P_1+P_2)/L. \tag{30}$$

From (30) it follows that the maximum bending moment at 2′ occurs when the average load on the left of the section is the same as the average load on the entire bridge. It will be seen that the criterion will be satisfied for a bridge loaded with equal joint loads when the bridge is fully loaded.

Uniform Loads.—In Fig. 88 the bending moment at 2′ due to a uniform load $p \cdot dx$ will be $p \cdot y \cdot dx$ in (a). But $y \cdot dx$ is the area of the influence diagram under the uniform load, and the bending moment at 2′ due to a uniform load will be equal to the area of the influence diagram covered by the load, multiplied by the load per unit of length. For a uniform load, p, covering the entire span the bending moment at 2′ will be p times the area of the influence diagram 1–2–3. For a uniform load the bridge must be fully loaded to obtain maximum bending moment at any point. It will be seen that the general criterion for maximum bending moment is satisfied when the bridge is fully loaded with a uniform load.

Maximum Shear in a Beam.—It is required to calculate the maximum shear at the point 2′ in the beam 1′–4′, in Fig. 89. Let P_1 represent a load on the left of the point, and P_2 a load on the right of the point 2′. Now with the load unity at 2′ the shear at the left of the point will be $(l-a)/l$, and on the right of the point 2′ the shear will

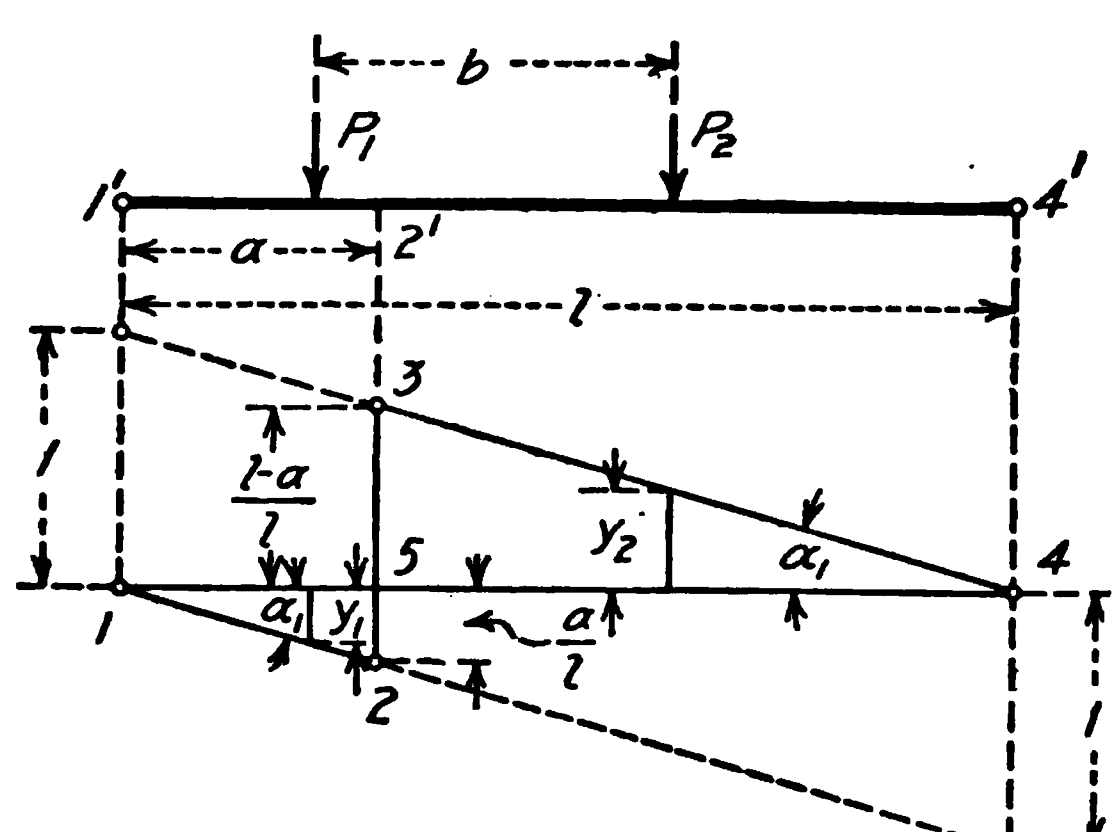


FIG. 89. INFLUENCE DIAGRAM FOR SHEAR IN A BEAM.

be $-a/l$, while the influence diagram for shear is 1–2–3–4. Then the shear at the point 2′ due to the loads P_1 and P_2 will be $S = -P_1 \cdot y_1 + P_2 \cdot y_2$. Now as the loads are moved to the left, the positive shear is increased and the negative shear is decreased. If the distance b is less than the distance a, for a maximum positive shear for two loads, P_2 should be at 2′, providing P_2 is greater than P_1. If P_1 is greater than P_2 the loads should be reversed in position.

For more than two loads, P_1 and P_2 representing the summations of the loads acting through the centers of gravity of the loads, the criterion is developed as follows:

The shear at 2′ is

$$S = -P_1 \cdot y_1 + P_2 \cdot y_2 \tag{31}$$

Now move the loads to the left a distance dx, no loads passing the point 2′ nor coming on nor going off the span, and

$$S + dS = -P_1(y_1 - dy_1) + P_2(y_2 + dy_2) \tag{32}$$

subtracting (31) from (32), and solving for a maximum, we have

$$dS = +P_1 \cdot dy_1 + P_2 \cdot dy_2 = 0 \tag{33}$$

Now $dy_1 = -dy_2$, and $P_1 = P_2$ is the criterion for maximum shear at 2′. In order to satisfy this criterion a load will come at 2′ which may be considered as a part of either P_1 or P_2.

Maximum Shear in a Truss.—Let P_1, P_2 and P_3, in Fig. 90, represent the loads on the left of the panel, on the panel, and on the right of the $(n+1)$st panel, respectively. It is required to find the position of the loads for a maximum shear in the panel.

With a load unity at 2′ the shear in the panel is $-m/n$, and 1–2 is the influence shear line for loads to the left of the panel. With a load unity at 3′ the shear in the panel is $(n-m-1)/n$, and the line 3–4 is the influence shear line for loads to the right of the panel. For a load on the panel the shear will vary from $-m/n$ at 2′ to $(n-m-1)/n$ at 3′, and the line 2–3 is the influence shear line for loads in the panel.

The influence diagram for the entire span is the polygon 1–2–3–4. It will be seen that the lines 1–2 and 3–4 are parallel, and are at a distance unity apart.

The total shear in the panel will then be

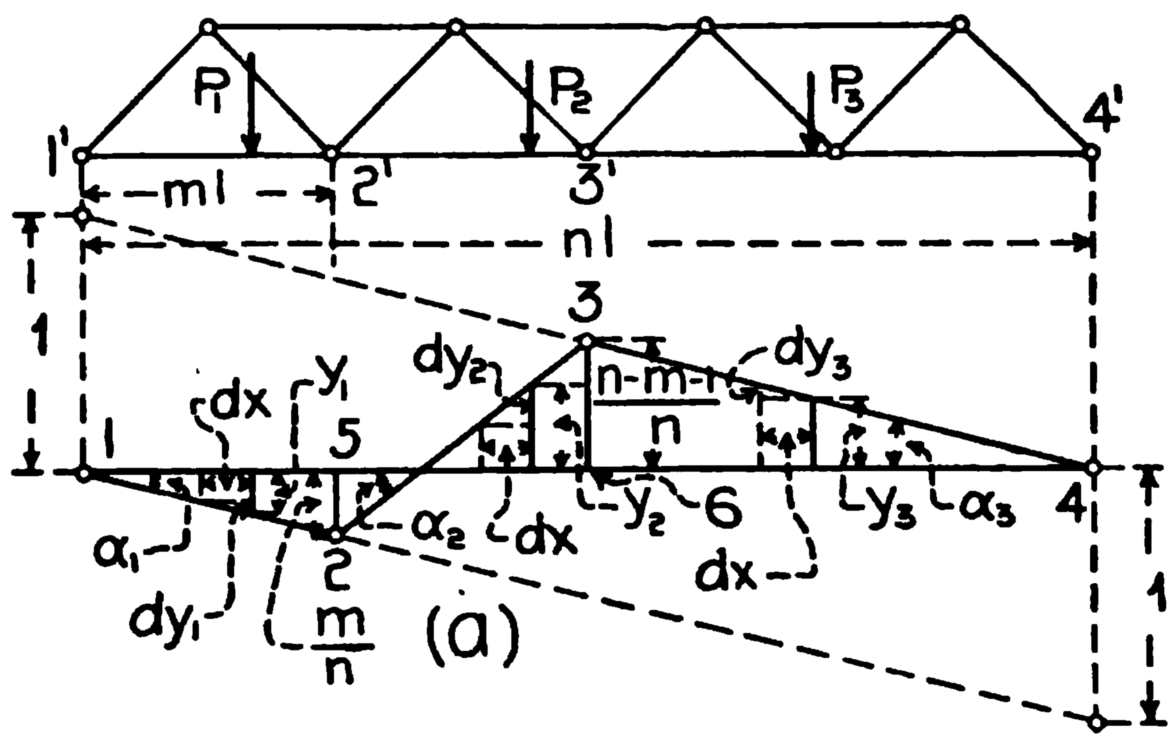


FIG. 90. INFLUENCE DIAGRAM FOR SHEAR IN A TRUSS.

$$S = -P_1 \cdot y_1 + P_2 \cdot y_2 + P_3 \cdot y_3 \qquad (34)$$

Now move the loads a short distance to the left, the distance being assumed so small that the distribution of the loads will not be changed, and

$$S + dS = -P_1(y_1 - dy_1) + P_2(y_2 - dy_2) + P_3(y_3 + dy_3) \qquad (35)$$

Subtracting (34) from (35), and solving for a maximum

$$dS = P_1 \cdot dy_1 - P_2 \cdot dy_2 + P_3 \cdot dy_3 = 0$$

But

$$dy_1 = dx \cdot \tan \alpha_1 = dx/n \cdot l,$$
$$dy_2 = dx \cdot \tan \alpha_2 = dx\,(n - 1)/n \cdot l,$$
$$dy_3 = dx \cdot \tan \alpha_3 = dx/n \cdot l;$$

and substituting we have

$$dS = P_1 \cdot dx/n \cdot l - P_2 \cdot dx(n - 1)/n \cdot l + P_3 \cdot dx/n \cdot l = 0$$
$$P_1 - P_2(n - 1) + P_3 = 0$$

and

$$P_1 + P_2 + P_3 = P_2 \cdot n$$

and

$$P_2 = (P_1 + P_2 + P_3)/n, \tag{36}$$

From (36) it follows that *the maximum shear in the panel will occur when the load on the panel is equal to the load on the bridge divided by the number of panels in the bridge.*

Uniform Loads.—In the same manner as for bending moment in Fig. 88, it can be proved that the shear in the panel due to a uniform load on the truss, in Fig. 90, is equal to the area of the influence diagram covered by the load, multiplied by the intensity of the uniform load per linear unit. From Fig. 90 it will be seen that for a uniform load the maximum shear in the panel will occur when the uniform load extends from the right abutment to that point in the panel where the line 2–3 passes through the line 1–4 (where the shear changes sign). For a minimum shear in the panel (maximum shear of the opposite sign) the load should extend from the left abutment to the point in the panel where the shear changes sign. For equal joint loads, load the longer segment for a maximum shear in the panel, and load the shorter segment for a minimum shear in the panel.

Maximum Floorbeam Reaction.—It is required to find the maximum load on the floorbeam at 2′ in (a) Fig. 91 for the loads carried by the floor stringers in the panels 1′–2′ and 2′–3′.

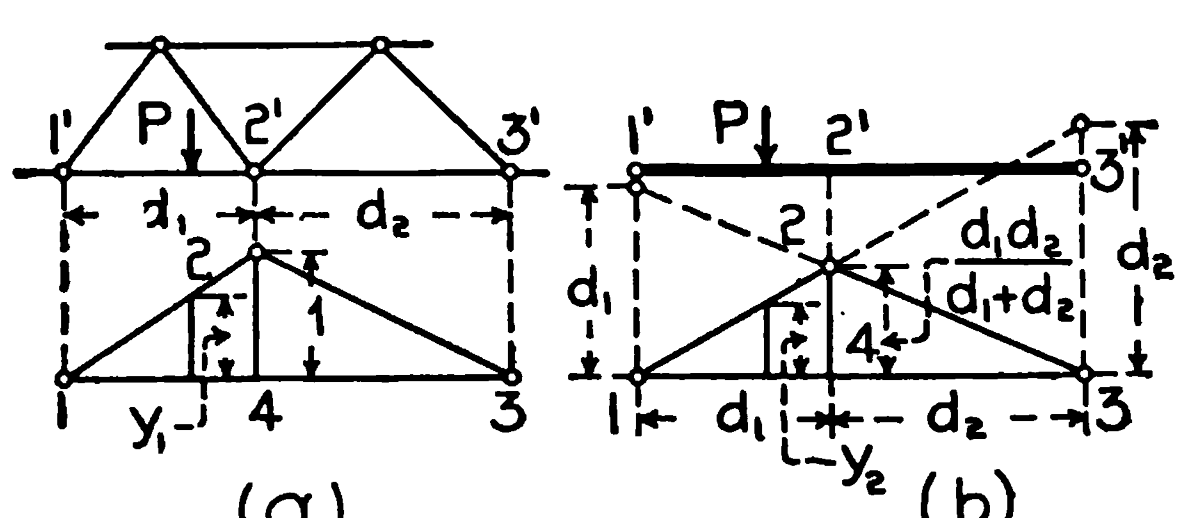


FIG. 91. INFLUENCE DIAGRAM FOR MAXIMUM FLOORBEAM REACTION.

In (a) the diagram 1–2–3 is the influence diagram for the shears at 2′ due to a unit load at any point in either panel. In (b) the dia-

gram 1–2–3 is the influence diagram for bending moment at 2′ for a unit load at any point in the beam. Now the diagram in (a) differs from the diagram in (b), only in the value of the ordinate 2–4. It will be seen that the reaction at 2′ in (a) may be obtained from the diagram in (b), for any system of loads, if the ordinates are multiplied by $(d_1 + d_2)/d_1 \cdot d_2$. We can therefore use diagram (b) for obtaining the maximum floorbeam reaction, if we multiply all ordinates by $(d_1 + d_2)/d_1 \cdot d_2$.

To obtain the maximum floorbeam reaction, therefore, take a simple beam equal to the sum of the two panel lengths, and find the maximum bending moment at a point in the beam corresponding to the panel point. This maximum moment multiplied by $(d_1 + d_2)/d_1 \cdot d_2$ *will be the maximum floorbeam reaction.* If the two panels are equal in length the maximum bending moment at the center of the beam multiplied by $2/d$, where d is the panel length, will give the maximum floorbeam reaction.

Maximum Moment in the Unloaded Chord of a Through Warren Truss.—Let P_1 in Fig. 92 represent the summation of the moving loads on the left of the panel 4′–5′, P_2 represent the summation of the moving loads on the panel, and P_3 represent the summation of the moving

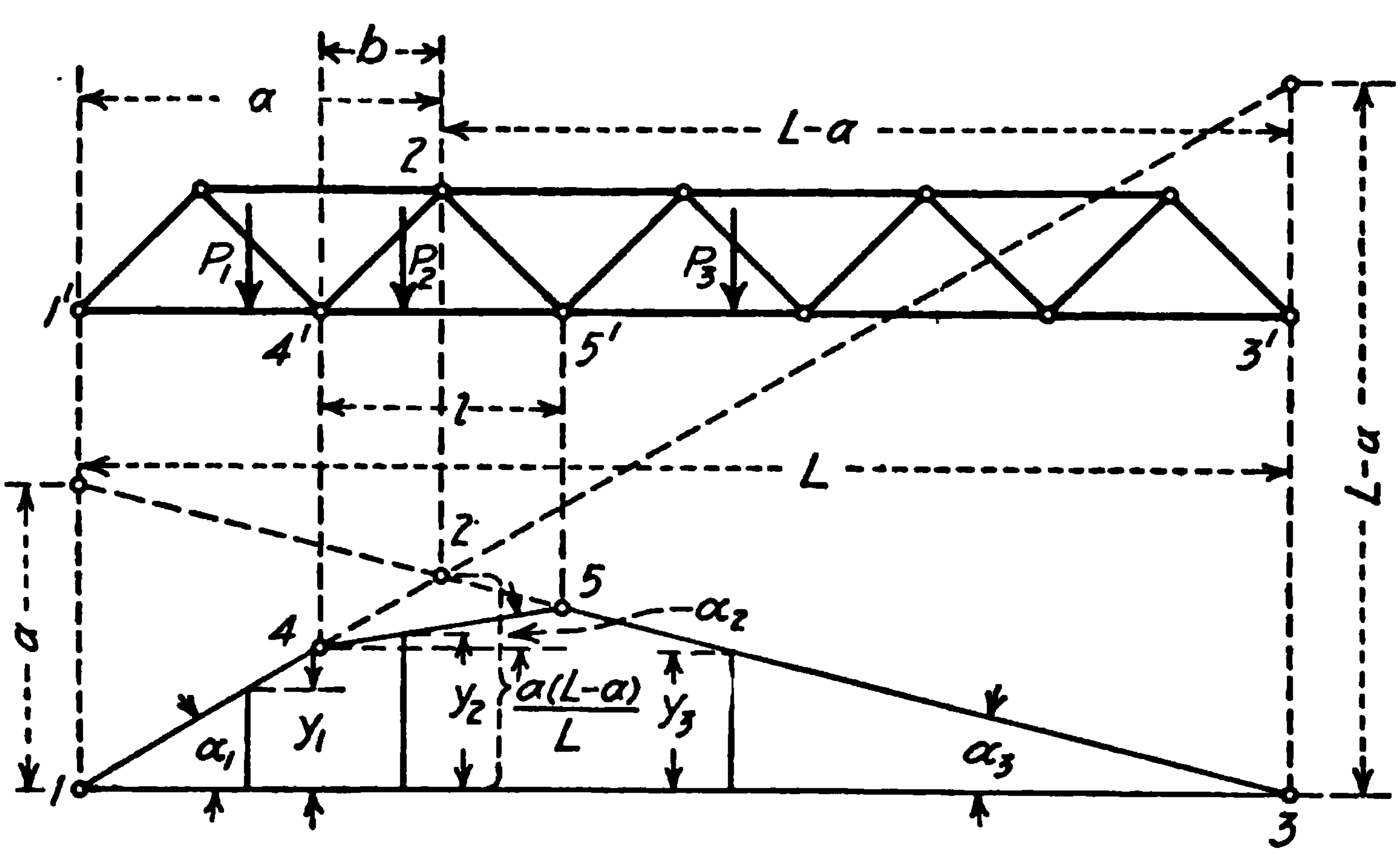


FIG. 92. INFLUENCE DIAGRAM FOR MOMENTS IN THE UNLOADED CHORD OF A THROUGH WARREN TRUSS.

loads to the right of the panel. The influence diagram for the point 2 is the diagram 1–4–5–3, the lines 1–4 and 5–3 are the same as for a point on the loaded chord, while the influence line for the panel 4′–5′ is the line 4–5.

Now the bending moment at 2 due to the three loads is

$$M = P_1 \cdot y_1 + P_2 \cdot y_2 + P_3 \cdot y_3 \tag{37}$$

Now move the loads P_1, P_2, P_3 a short distance to the left, the distance being assumed so small that the distribution of the loads will not be changed, and

$$M + dM = P_1(y_1 - dy_1) + P_2(y_2 - dy_2) + P_3(y_3 + dy_3) \tag{38}$$

Subtracting (37) from (38), and solving for a maximum

$$dM = -P_1 \cdot dy_1 - P_2 \cdot dy_2 + P_3 \cdot dy_3 = 0 \tag{39}$$

Now $dy_1 = dx \cdot \tan \alpha_1$, $dy_2 = dx \cdot \tan \alpha_2$, and $dy_3 = dx \cdot \tan \alpha_3$.

$\tan \alpha_1 = (L - a)/l$, $\tan \alpha_3 = a/L$, and $\tan \alpha_2$

$$= \frac{[L - (a - b + l)] \tan \alpha_2 - (a - b) \tan \alpha_1}{l}$$

and $\tan \alpha_2 = (L \cdot b - a \cdot l)/L \cdot l$.

Substituting the values of $\tan \alpha_1$, $\tan \alpha_2$ and $\tan \alpha_3$ in (39) we have

$$-P_1(L - a)/L - P_2(L \cdot b - a \cdot l)/L \cdot l + P_3 \cdot a/L = 0$$

Solving and placing $P = P_1 + P_2 + P_3$, we have

$$P/L = (P_1 \cdot l + P_2 \cdot b)/a \cdot l \tag{40}$$

Equation (40) is the criterion required.

Maximum Stresses in a Bridge with Inclined Chords.—Let $U_2 4'$ be a web member in a truss with inclined chords in Fig. 93. Point A is the intersection of the upper chord $U_1 U_2$ and the lower chord 2′4′. The stress in $U_2 4'$ equals the moment of the external forces about the point A, divided by the arm c. The stress in the web member $U_2 4'$ will then be a maximum when the bending moment at A is a maximum. To draw the moment influence diagram for the point A, calculate the bending moments about A for the unit loads at 2′ and 4′. With a load

unity at $4'$ the moment at A is $(L-a-l)e/L$, and with a load unity at $2'$ the moment at A is $(L-a)e/L-(a+e)$, a negative quantity. Laying off 4–6 and 2–7 equal to these moments, the influence diagram for bending moment at A is the polygon 1–2–4–5.

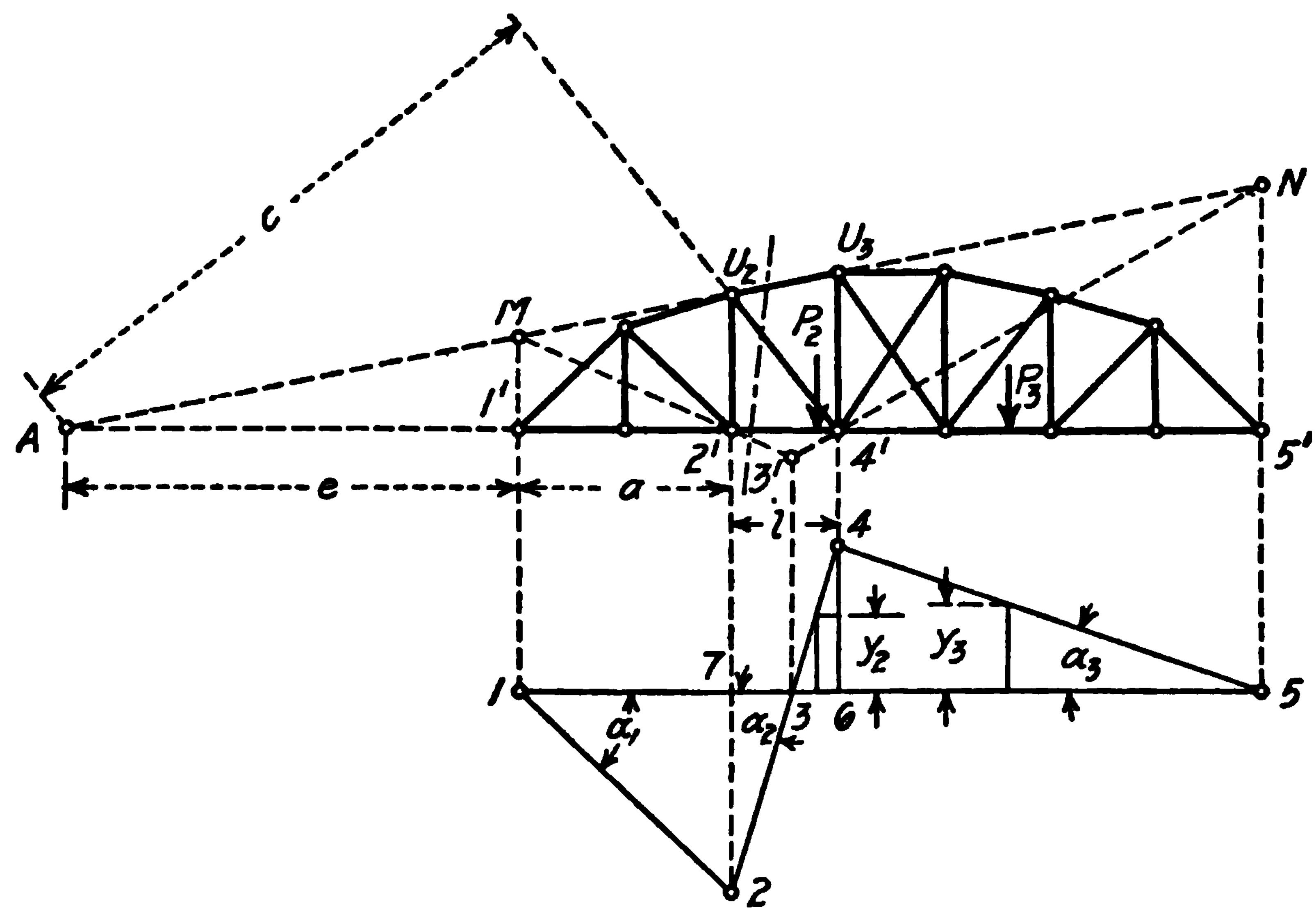


Fig. 93.

The maximum stress in $U_2 4'$ occurs when some of the wheels at the head of the train are in the panel $2'4'$, and in unusual cases only, when a load is to the left of $2'$. Load P_2 representing the summation of the loads to the left of $4'$ will always come in the panel $2'4'$. Load P_3, representing the summation of the loads to the right of the panel, will always come to the right of the panel $2'4'$. Now the moment at A is

$$M = P_2 \cdot y_2 + P_3 \cdot y_3 \tag{41}$$

Now move the loads a differential distance to the left, it being assumed that the distribution of the loads is not changed, and

$$M + dM = P_2(y_2 - dy_2) + P_3(y_3 + dy_3) \tag{42}$$

Subtracting (41) from (42), and solving for a maximum we have

$$dM = -P_2 \cdot dy_2 + P_3 \cdot dy_3 = 0 \tag{43}$$

Now $dy_2 = dx \cdot \tan \alpha_2$, and $dy_3 = dx \cdot \tan \alpha_3$, and

$$-P_2 \cdot \tan \alpha_2 + P_3 \cdot \tan \alpha_3 = 0 \tag{44}$$

and if $P = P_2 + P_3$,

$$P/(5\text{–}3) = P_2/(3\text{–}6) \tag{45}$$

Now $\tan \alpha_2 = -(l \cdot e/L - a - e)/l$, and $\tan \alpha_3 = e/L$, and substituting in (44) we have

$$P/L = P_2(1 + a/e)/l \tag{46}$$

Now for a uniform load the maximum stress in the member $U_2 4'$ will occur when the truss is loaded from the right abutment to the point 3′, while the minimum stress will occur when the load extends from the left abutment to the point 3′. The critical point 3 can be calculated by drawing the lines M–2′–3′ and N–4′–3′. For wheel loads no load, should in general, pass 3′ from the right to give a maximum stress in the member.

By substituting $e = \infty$ in (46) we have the criterion for maximum shear in a panel of a bridge with parallel chords.

Resolution of the Shear.—In Fig. 94 the stresses U, D and L hold in equilibrium the external forces on the left of the section cutting these members. These external forces consist of a left reaction, R, at the left abutment and a force at 2, equal to the reaction of the stringer 2–3. The resultant, S, of these two forces acts at a point a little to the left of the left reaction. Its position may be determined by moments. Referring to Fig. 94, let the resultant, S, be replaced by the two forces P_1 and P_3, P_1 acting upwards at 1 and P_3 acting downward at 3 as shown. Now taking moments about point 1, and

$$S \cdot a = P_3 \cdot l \tag{47}$$

Now the bending moment at 1 equals M_1, and

$$P_3 = S \cdot a/l = M_1/l \tag{48}$$

Similarly by taking moments at 3, we have

$$S(a+l)=P_1 \cdot l, \text{ but } S(a+l)=M_3, \text{ and } P_1=M_3/l$$

Now

$$S=P_1-P_2=M_1/l-M_3/l \qquad (49)$$

where S is the shear in the panel.

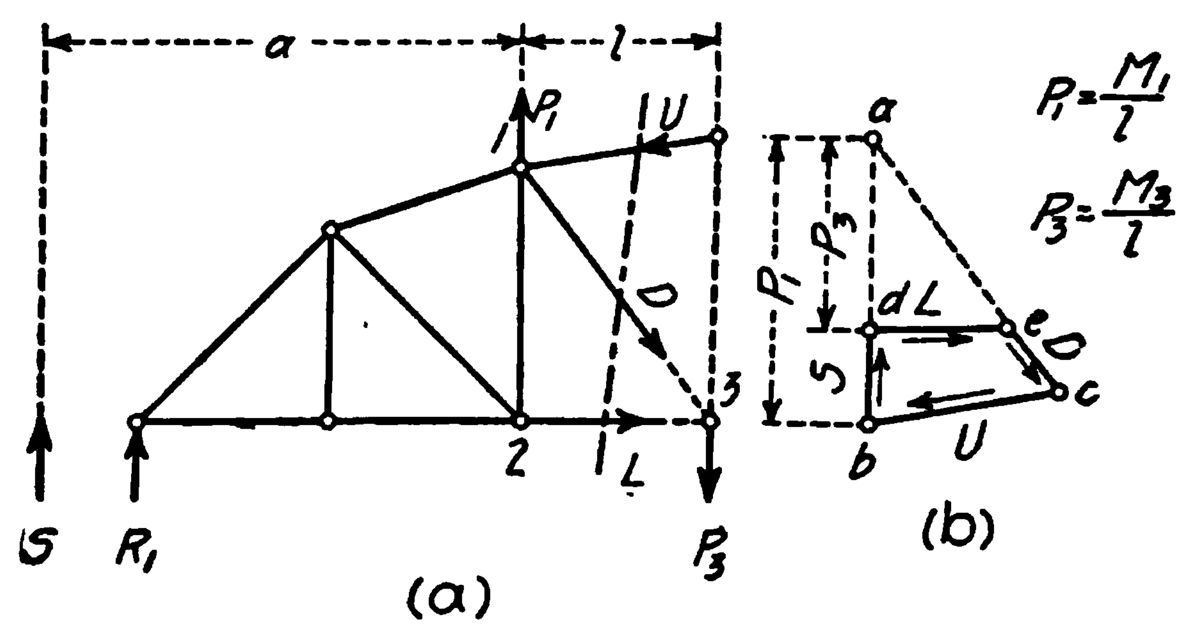


FIG. 94.

Moment Diagram.—A moment diagram for Cooper's Class E 40 loading is given in Fig. 95. The bending moments are given in thousands of foot-pounds for one rail. The first line gives the summation of the weights of the wheels in thousands of pounds, calculated from the head of the uniform load; the second line gives the summation of the weights of the wheels, calculated from wheel 1 in the leading locomotive; the number of the wheel from the left is given in the small circle in each wheel; the weight of each wheel in thousands of pounds is given in each wheel; the first line under the wheels gives the distances in feet between the centers of the wheels; the second line under the wheels gives the summation of the distances from wheel 1; while the third line under the wheels gives the summation of the distances from the head of the uniform load. The fourth line below the wheels gives the summation of the moments of all wheels to the left about the head of the uniform load in thousands of foot-pounds for one rail. For example, the moment of all the wheels about the head of the uniform load is 16,364 thousand foot-pounds. The fifth line below the wheels gives the summation of the moments of all wheels up to and including the wheel on the left about wheel 18, etc. Below the stepped line the moments of the wheels to the right of the stepped

MOMENT DIAGRAM FOR COOPER'S CLASS E40 LOADING.

In thousand foot-pounds for one rail.

284	274	254	234	214	194	181	168	155	142	132	112	92	72	52	39	26	13
10	30	50	70	90	103	116	129	142	152	172	192	212	232	245	258	271	284
10 (1)	20 (2)	20 (3)	20 (4)	20 (5)	13 (6)	13 (7)	13 (8)	13 (9)	10 (10)	20 (11)	20 (12)	20 (13)	20 (14)	13 (15)	13 (16)	13 (17)	13 (18)
8'	5'	5'	5'	9'	5'	6'	5'	8'	8'	5'	5'	5'	9'	5'	6'	5'	5'
8'	13'	18'	23'	32'	37'	43'	48'	56'	64'	69'	74'	79'	88'	93'	99'	104'	109'
109'	101'	96'	91'	86'	77'	72'	66'	61'	53'	45'	40'	35'	30'	21'	16'	10'	5'
16364	15274	13254	11334	9514	7794	6793	5857	4999	4206	3676	2776	1976	1276	676	403	195	65
14944	13904	11984	10164	8444	6824	5888	5017	4224	3496	3016	2216	1516	916	416	208	65	
13589	12599	10779	9059	7439	5919	5048	4242	3514	2851	2421	1721	1121	621	221	78	65	
12041	11111	9411	7811	6311	4911	4118	3390	2740	2155	1785	1205	725	345	65	78	221	
10816	9936	8336	6836	5436	4136	3408	2745	2160	1640	1320	840	460	180	65	208	416	
8728	7938	6518	5198	3978	2858	2247	1701	1233	830	600	300	100	117	299	559	884	
7668	6928	5608	4388	3268	2248	1702	221	818	480	300	100	100	282	529	854	1244	
6708	6018	4798	3678	2658	1738	1257	841	503	230	100	100	300	547	859	1249	1704	
5848	5208	4088	3068	2148	1328	912	561	288	80	100	300	600	912	1289	1747	2264	
4632	4072	3112	2252	1492	832	520	273	104	160	420	780	1240	1656	2137	2696	3320	

Uniform Load

2000 lbs. per lin. ft.

FIG. 95.

line are given in like manner. For example, it is required to calculate the maximum bending moment at panel point L_1 in a 6-panel Pratt truss, having a span of 150 feet, it having been determined by applying the criterion above that wheel 4 at L_1 produces a maximum bending moment at L_1. Wheel 4 is 18 feet from wheel 1, and panel L_1 is 125 feet from the right abutment. The distance from wheel 1 to the right abutment is then $18 + 125 = 143$ feet. The distance from wheel 1 to the head of the uniform load is 109 feet, and there is $143 - 109 = 34$ feet of uniform load on the bridge. The moment of the two locomotives about the head of the uniform load is 16,364 thousand foot-pounds, and the moment of the locomotives about the right abutment is $16{,}364 + 284 \times 34 = 16{,}364 + 9{,}656 = 26{,}020$ thousand foot-pounds. The moment of the uniform load about the right abutment is $34 \times 2 \times 34/2 = 34^2 = 1156$ thousand foot-pounds. The total bending moment about the right abutment then is $16{,}364 + 284 \times 34 + 34^2 = 27{,}176$ thousand foot-pounds.

The left reaction is, $R_1 = 27{,}176/150 = 181.17$ thousand pounds. The moment of the left reaction about the panel point $L_1 = 27{,}176/150 \times 25$ thousand foot-pounds. The bending moment of wheels 1, 2 and 3 about wheel 4 is, from the moment diagram, 480 thousand foot-pounds. The bending moment at panel point L_1 will then be

$$= 27{,}176/150 \times 25 - 480 = 4{,}049.3$$

thousand foot-pounds. To calculate the stress in the lower chord members L_0L_1 and L_1L_2, divide the bending moment just obtained by the depth of the truss.

The shear in the panel L_0L_1 is equal to the left reaction of the truss minus the left reaction of the floor stringers, is $S = 181.17 - 480/25 = 181.17 - 19.2 = 162.17$ thousand pounds.

For the calculation of the stresses in a Pratt truss, see Problem 21, Chapter IX.

CHAPTER VII.

Stresses in Lateral Systems.

Introduction.—The wind loads on bridges are carried to the abutments by the lateral systems. In a through truss bridge the lateral systems usually consist of the top lateral system, the bottom lateral system, the intermediate bents or sway bracing between the intermediate posts, and the portals in the planes of the end-posts as shown in Fig. 1. In shallow through truss bridges the sway bracing is sometimes omitted; in deck trusses the portals are replaced by sway bracing; while in low trusses the bottom lateral system only is used.

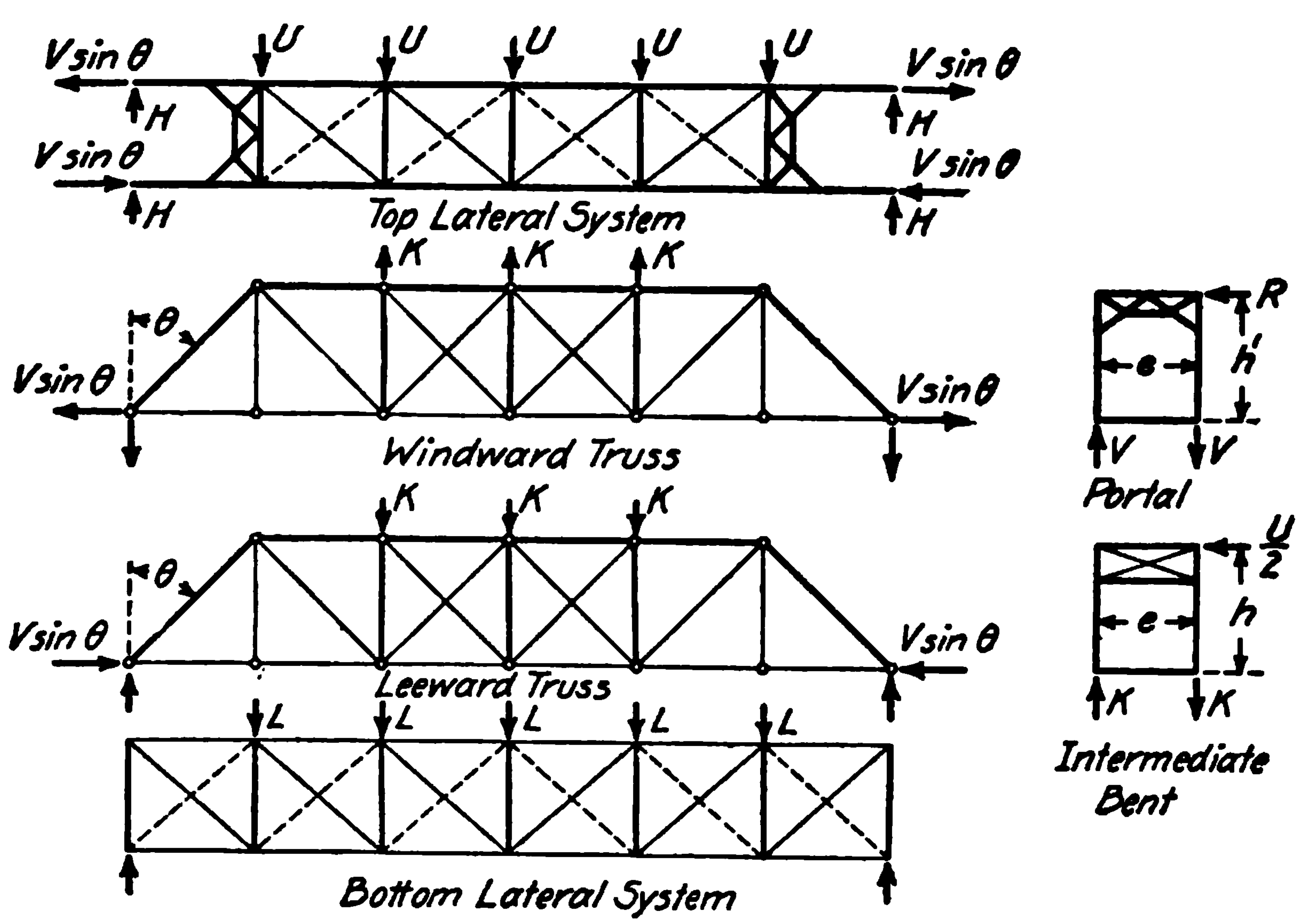


Fig. 96.

Wind Loads.—The wind loads are usually given in specifications as a certain number of pounds per lineal foot of bridge or per square foot of exposed surface. The wind load is usually taken at 30 lbs.

per square foot of exposed surface when the live load is on the bridge and at 50 lbs. per square foot of exposed surface when the bridge is unloaded.

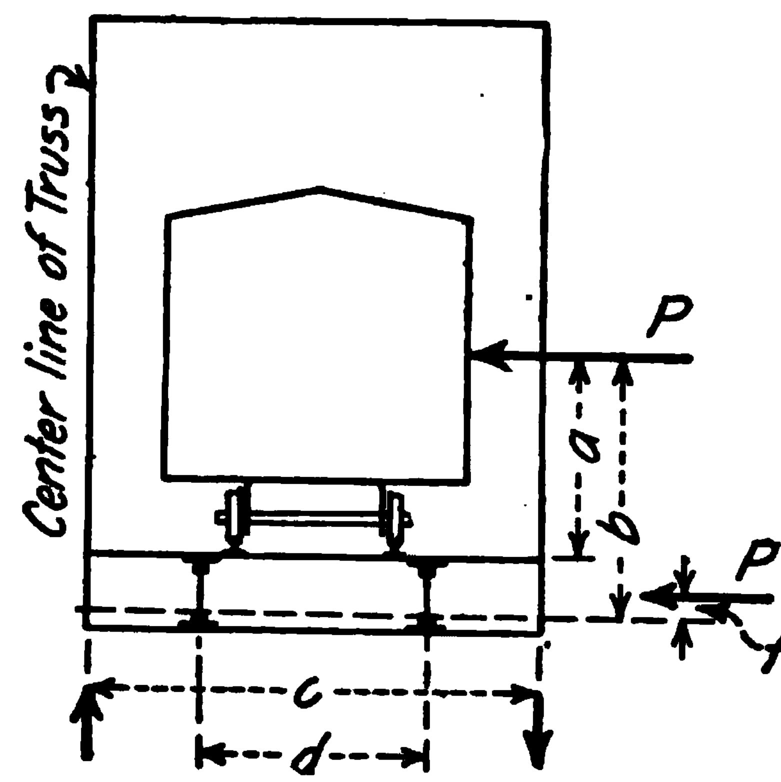


FIG. 97a.

The usual specification for highway bridges is: A wind load of 150 lbs. per lineal foot of bridge on the unloaded chord to be treated as a dead load, and a wind load of 300 lbs. per lineal foot of bridge on the loaded chord, 150 lbs. of which is to be treated as a dead load and 150 lbs. to be treated as a live load. In railroad bridges the dead load wind is usually taken the same as for highway bridges, while the live load wind is taken at 450 lbs. per lineal foot. For extracts from standard specifications, see Chapter II.

STRESSES IN LATERAL SYSTEMS.—In the through Pratt truss bridge in Fig. 96 the wind joint loads on the upper chord are equal to U, while the joint loads on the lower chord are equal to L. Where sway bracing is used, part of the upper chord loads are transferred to the lower lateral system by the sway bracing. The exact amount thus transferred is statically indeterminate, but is usually assumed as $U/2$ at all joints having sway bracing. This load $U/2$ produces a vertical load, K, at each joint in the vertical trusses, which acts downward on the leeward and upward on the windward side of the bridge. Each portal transfers the load carried to the hip joint by the upper lateral system, and the load at the hip joint to the abutments.

This produces a tension $V \cdot \sin\theta$ in the bottom chord on the leeward side and a compression $V \cdot \sin\theta$ in the bottom chord on the windward side.

The stresses in the lateral systems of a highway bridge are calculated in detail in Fig. 286, in Part III.

In addition to the wind loads on the top chord that are transferred to the bottom lateral system, the wind load on the train of cars on steam and electric railway bridges, increases the loading on the vertical trusses on the leeward side and decreases the loading on the windward side of the bridge as shown in Fig. 97a. This increase or decrease in vertical loading can be calculated by taking moments about the line of the pins in the lower chord. The wind load acting on the train is usually specified as applied six feet above the base of the rail.

Skew Bridge.—In a skew bridge the abutments are not at right angles to the center line of the bridge. This gives a warped portal, and

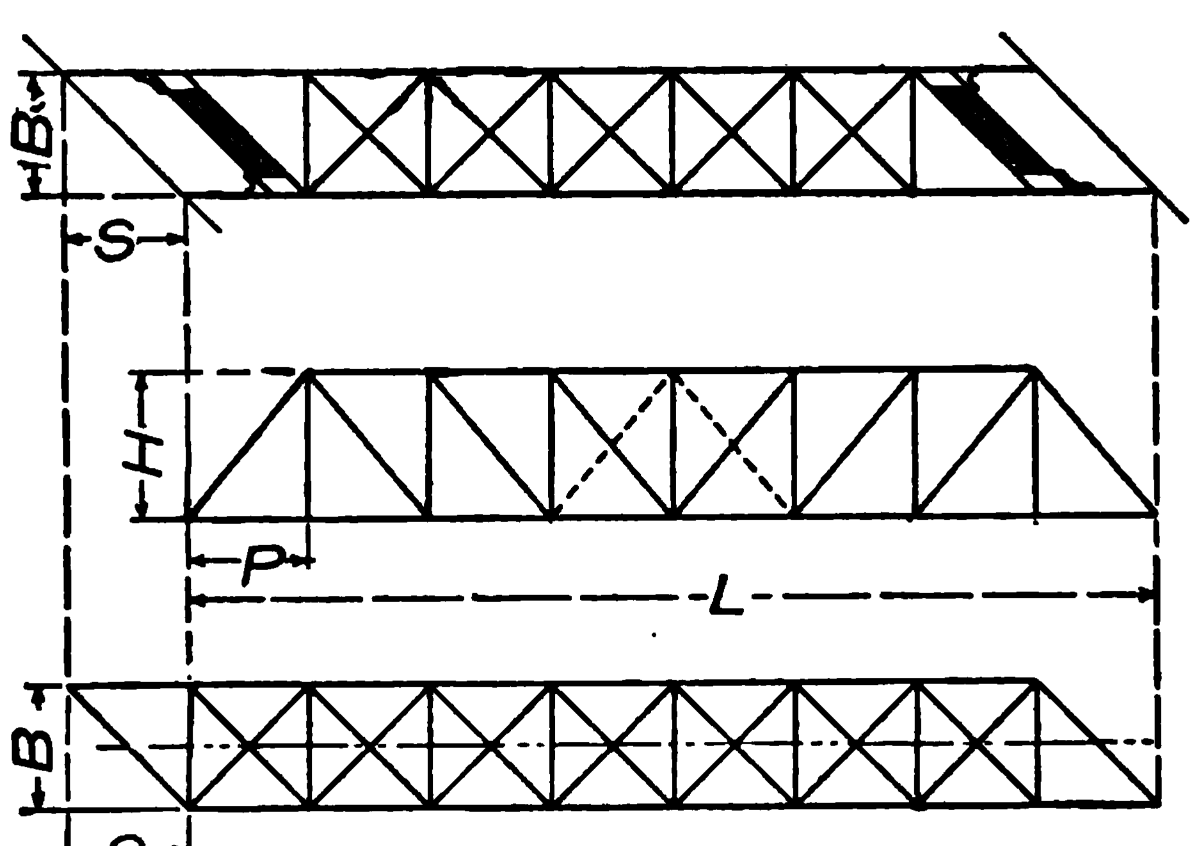


FIG. 97*b*. SKEW BRIDGE.

ties which are not vertical unless the bridge is skewed one entire panel as is shown in Fig. 97*b*, which is the common practice.

Initial Stresses.—In (a) Fig. 98 the diagonal lateral rods have an initial stress of 10,000 lbs. in each rod. In (b) the lateral truss is loaded with loads of 12,000 lbs. at joints *B*, *C* and *D*, producing stresses

as shown. In (c) the combined stresses due to direct loads and the initial stresses are shown. The stresses in the chords and struts can now be calculated by algebraic resolution. The stresses are combined as follows: In panel *B–C* each rod has an intitial stress of 10,000 lbs., and in addition must transfer a wind shear of 6,000 lbs. or an inclined stress of 9,000 lbs. Half of the 9,000 lbs. or 4,500 lbs. will be added

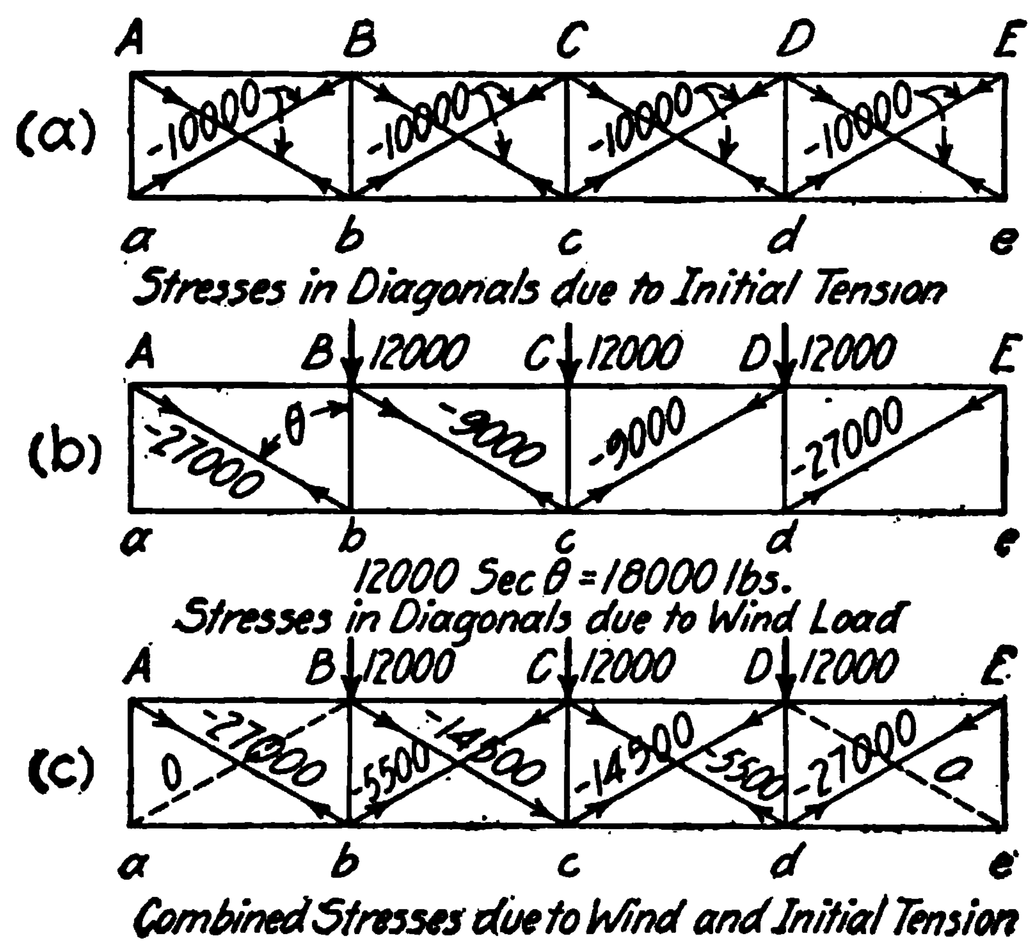


FIG. 98.

to the initial stress in *Bc,* making the stress — 14,500 lbs., while 4,500 lbs. will be subtracted from the initial stress in *Cb,* making the stress — 5,500 lbs. In panel *A–B* the initial stress in *aB* is entirely relieved by the direct stress, while the initial stress in *Ab* is increased by the direct stress remaining after the initial stress of 10,000 lbs. was relieved in *aB,* or 17,000 lbs., making the total stress in *Ab* = — 10,000 — 17,000 = — 27,000 lbs., the same as if there had been no initial stress in the panel. This solution is based on the mathematical principle "*That if a load may be carried from one point to another by more than one route, it will be divided between the routes in proportion to the rigidities of the routes.*" In the problem above the two routes are assumed to have the same rigidities.

PORTALS.—Portal bracing is placed at the ends of through

bridges in the planes of the end-posts to transfer the wind loads from the upper lateral system to the abutments. The stresses in the sway bracing placed in the planes of the intermediate posts are calculated in the same manner as the stresses in portal bracing. Portal bracing should be designed so that the stresses will be statically determinate. Several different types of portals are shown in Fig. 99. Types (a),

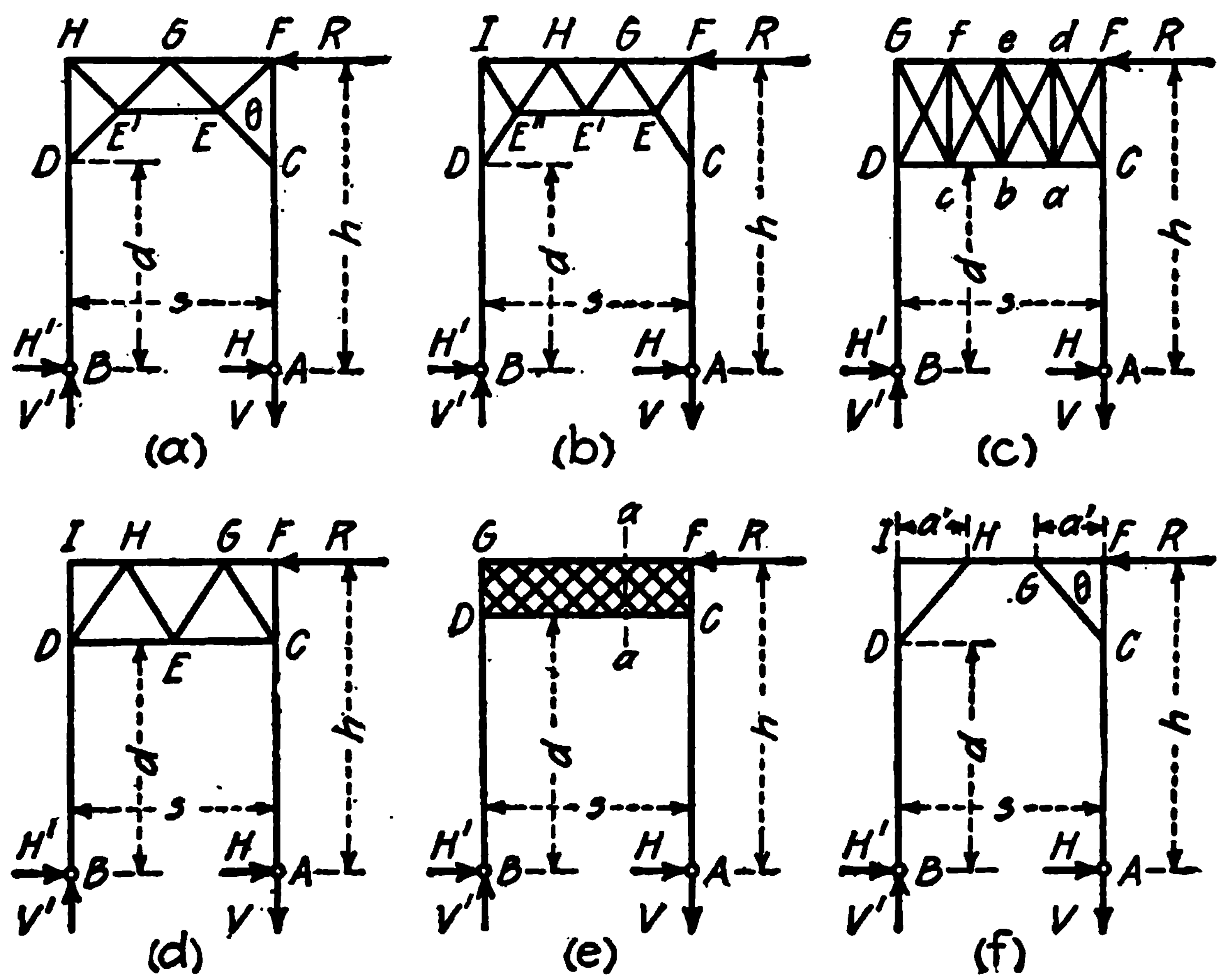


FIG. 99. TYPES OF PORTALS.

(b) and (d) are the ones most used for highway bridges. The lower ends of the end-posts may be hinged (free to turn), or fixed. The criterion for determining whether the end-posts are fixed or not will be discussed in Chapter VII.

Case I. Stresses in Simple Portals: *End-posts Hinged.*—The deflections of the posts in the portals shown in Fig. 99 are assumed to be equal, and

$$H = H' = R/2$$

Taking moments about the foot of the windward post

$$V' = - V = R \cdot h/s$$

Having found the external forces, the stresses in the members may be found by either algebraic or graphic methods.

Algebraic Solution. *Portal* (*a*).—To obtain the stress in member G–C, (a) Fig. 99, pass a section cutting G–F, E–F and G–C, and take moments of the external forces to the right of the section, about point F as a center.

$$G\text{–}C = - H \cdot h/[(h - d) \sin \theta] \tag{50}$$

But $H = R/2$, and $(h - d) \sin \theta = \frac{1}{2} s \cdot \cos \theta$. Substituting these values in (50) we have

$$G\text{–}C = - R \cdot h/(s \cdot \cos \theta) = - V \cdot \sec \theta \tag{51}$$

Resolving at C and F we have, stress in E–$F = 0$, and also stresses E–E' and H–$E' = 0$.

To obtain stress in G–D, pass section cutting H–G, H–E' and G–D, and take moments of the external forces to the left of the section, about point H as a center.

$$G\text{–}D = H \cdot h/[(h - d) \sin \theta] = + V \cdot \sec \theta \tag{52}$$

To obtain stress in G–F, pass a section cutting G–F, E–F and G–C, and take moments of the external forces to the right of the section, about point C as a center.

$$G\text{–}F = + [R(h - d) + H \cdot d]/(h - d) \tag{53}$$

To obtain stress in H–G, pass a section cutting H–G, H–E' and G–D, and take moments of the external forces to the left of the section, about the point D as a center.

$$H\text{–}G = - H \cdot d/(h - d) \tag{54}$$

The stress in the windward post, A–F, is zero above and V below the foot of the knee brace C; the stress in the leeward post is zero above and V' below the foot of the knee brace D.

The shear in the posts is H below the foot of the knee brace, and above the foot of the knee brace is given by the formula

$$S = H \cdot d/(h - d) = \text{stress in } H\text{–}G \tag{55}$$

The maximum moment in the posts occurs at the foot of the knee braces C and D, and is

$$M = H \cdot d \tag{56}$$

For the actual stresses, moments and shears in a portal of this type, see Fig. 100.

Portal (*b*).—The stresses in portal (b) Fig. 99, are found in the same manner as in portal (a). The graphic solution of a similar portal with one more panel is given in Fig. 101, which see. It should be noted that all members are stressed in portals (b) and (d).

Portal (*c*).—The stresses in portal (c) Fig. 99, may be obtained (1) by separating the portal into two separate portals with simple bracing, the stresses found by calculating the separate simple portals with a load $= \frac{1}{2}R$, being combined algebraically, to give the stresses in the portal; or (2) by assuming that the stresses are all taken by the system of bracing in which the diagonal ties are in tension. The latter method is the one usually employed and is the simpler.

Maximum moment, shear and stresses in the posts are given by the same formulas as in (a) Fig. 99.

Portal (*e*).—In portal (e) Fig. 99, the flanges G–F and D–C are assumed to take all the bending moment, and the lattice web bracing is assumed to take all the shear. The maximum compression in the upper flange G–F occurs at F, and is

$$G\text{–}F = + [R(h - d) + H \cdot d]/(h - d) \tag{57}$$

The maximum tension in the upper flange G–F is

$$G\text{–}F = - H \cdot d/(h - d) \tag{58}$$

The maximum stress in the lower flange D–C is

$$D\text{–}C = \pm H \cdot h/(h - d) \tag{59}$$

maximum tension occurring at C, and maximum compression occurring at D.

The maximum shear in the portal strut is V, which is assumed as taken equally by the lattice members cut by a section, as a–a.

Maximum moment, shear and stresses in the posts are given by the same formulas as in (a) Fig. 99.

Portal (f).—The maximum moment in the portal strut I–F in (f) Fig. 99, occurs at H and G, and is

$$M = + H \cdot h - V \cdot a \tag{60}$$

The maximum direct stress in H–G is $+H$, and in I–H is

$$I\text{–}H = - H \cdot d/(h - d) \tag{61}$$

The maximum stress in G–F is given by formula (53).

The maximum shear in girder I–F is equal to V. The stress in G–C is $-V \cdot \sec\theta$ and in H–D is $+V \cdot \sec\theta$, as in (a) Fig. 99.

Portal strut I–F is designed as a girder to take the maximum moment, shear and direct stress.

Maximum moment, shear and stresses in the posts are given by the same formulas as in (a) Fig. 99.

Graphic Solution.—To make the solution of the stresses statically determinate, replace the posts in the portals with trussed framework as in Fig. 100. The stresses in the interior members are not affected by substituting the dotted members, and will be correctly given by graphic resolution.

As before $H = H' = R/2$ and $V = -V' = R \cdot h/s$.

Having the calculated H, H', V and V', the stresses are calculated by graphic resolution as follows: Beginning at the base of the column A, lay off A–$4 = V = 3{,}000$ lbs. acting downward, and A–$a = H = 1{,}000$ lbs. acting to the right. Then a–1 and 4–1 are the stresses in members a–1 and 4–1, respectively, heavy lines indicating compression and light lines tension. At joint in auxiliary truss to right of C the stress in 1–a is known and stresses in 1–2 and 2–a are found by closing the polygon. The stresses in the remaining members are found in like manner, taking

joints C, E, F, etc., in order, and finally checking up at the base of the post B. The full lines in the stress diagram represent stresses in the portal; the dotted lines represent stresses in the auxiliary members or stresses in members due to auxiliary members, and are of no consequence. The shears and moments are shown in the diagram.

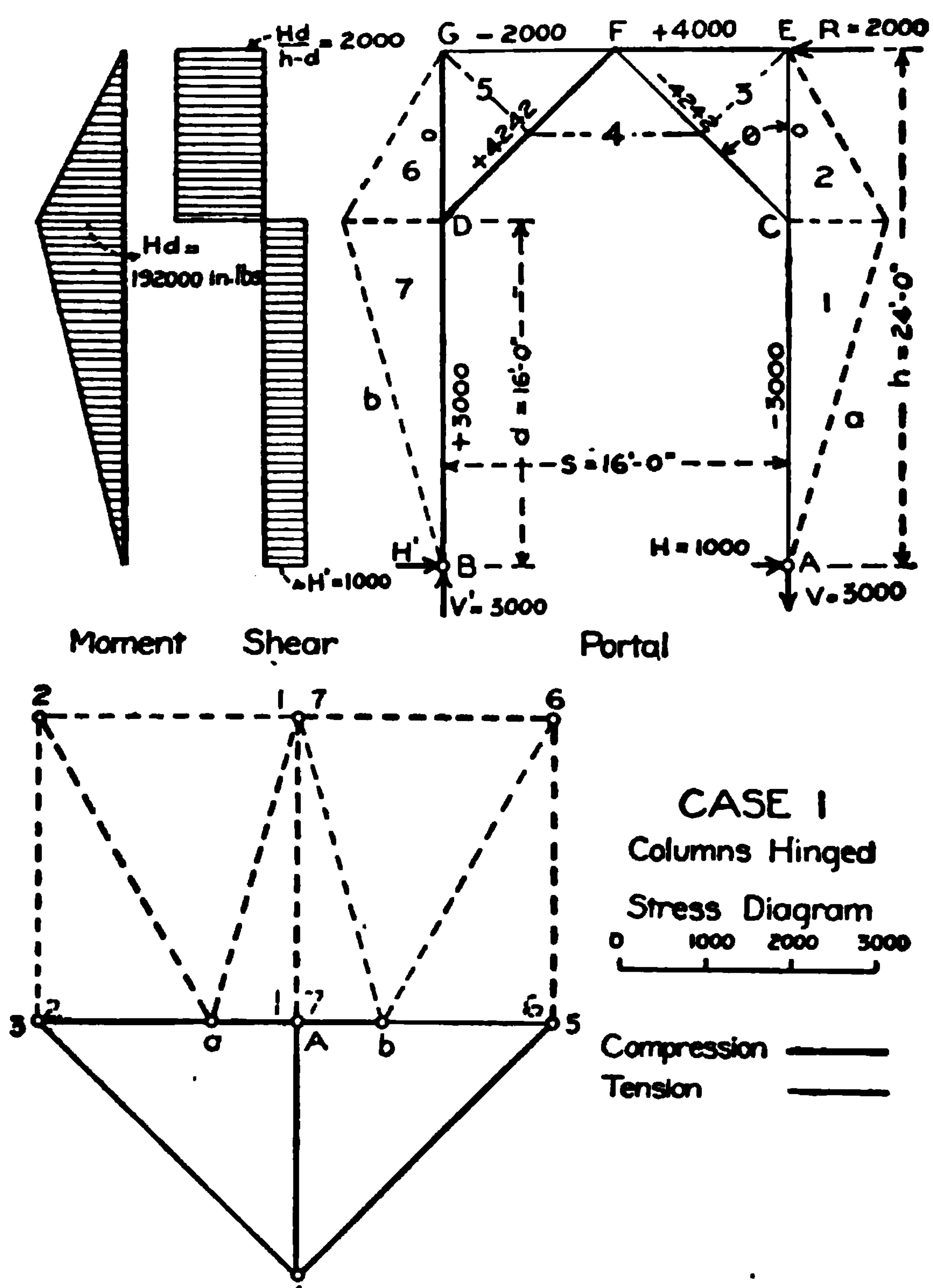


FIG. 100.

Simple Portal as a Three-Hinged Arch.—In a simple portal the resultant reactions and the external load, R, meet in a point at the middle of the top strut, and the portal then becomes a three-hinged arch ("Design of Steel Mill Buildings," Chapter XIII), provided there is a joint at that point (point b, Fig. 101).

In Fig. 101 the reactions were calculated graphically and the stresses in the portal were calculated by graphic resolution. Full lines in the

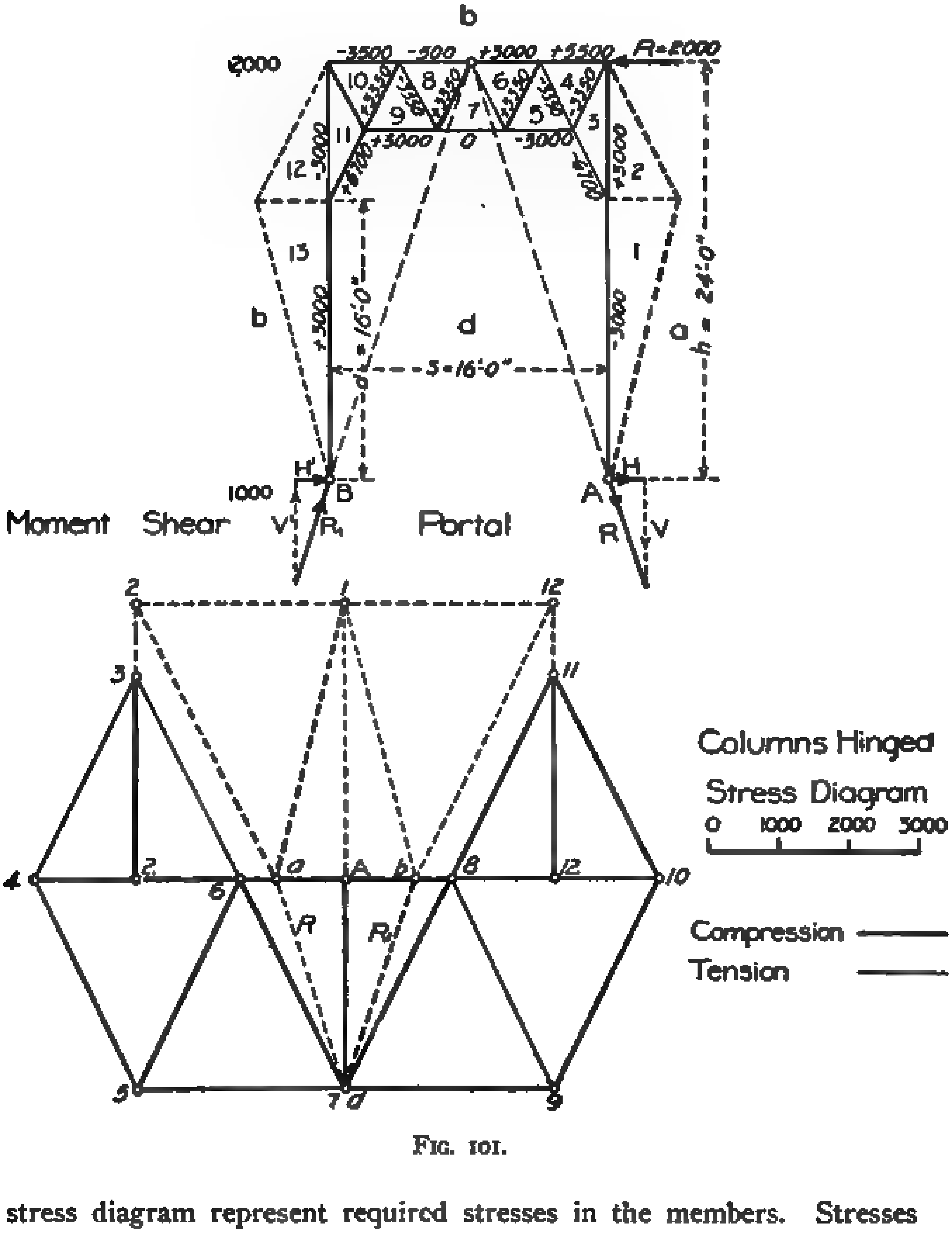


FIG. 101.

stress diagram represent required stresses in the members. Stresses 3–2 and 11–12 were determined by dropping verticals from points 3 and 11 to the load line 4–10.

Case II. Stresses in Simple Portals. *Posts Fixed.*—The calculation of the stresses in a portal with posts fixed at the base is similar

to the calculation of stresses in a transverse bent with columns fixed at the base.* The point of contra-flexure is at the point

$$y_0 = (d/2)\,\frac{d + 2h}{2d + h} \tag{62}$$

measured up from the base of the post. The point of contra-flexure is usually taken at a point a distance $d/2$ above the bases of the posts.

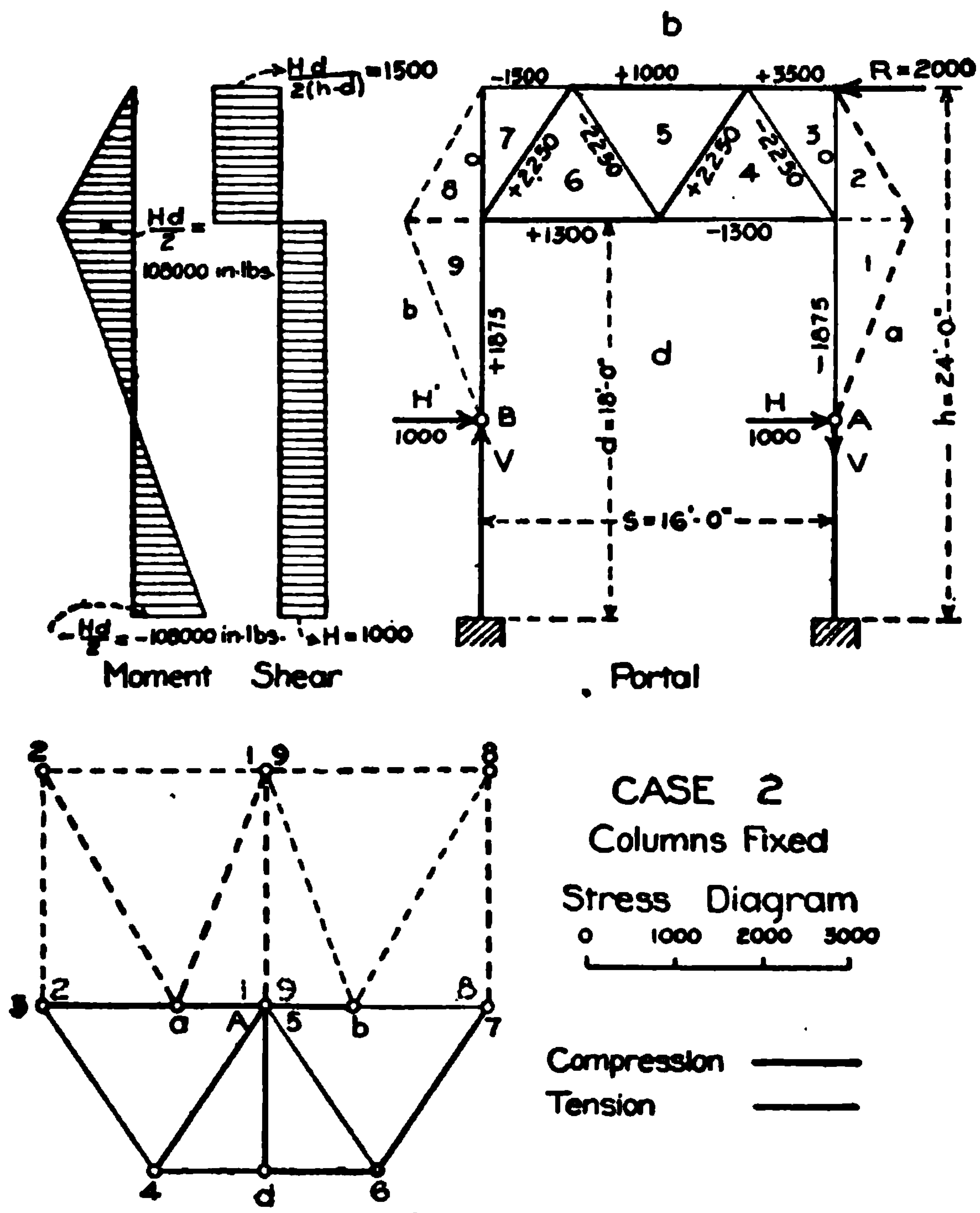


FIG. 102.

The stresses in a portal with posts fixed may be calculated by considering the posts hinged at the point of contra-flexure and solving as in Case I.

Algebraic Solution.—In Fig. 102 we have

$$H = H' = R/2$$

and

* See the author's book "The Design of Steel Mill Buildings," Chapter XI.

$$V = -V' = \frac{R(h-d)}{2s}$$

Having found the reactions H and H', V and V', the stresses in the members are found by taking moments as in (a) Fig. 99, considering the posts as hinged at the point of contra-flexure. The shear diagram for the posts is as shown in (a) and the moment diagram as in (c) Fig. 102.

Graphic Solution.—The stresses in the portal in Fig. 102 have been calculated by graphic resolution. This problem is solved in the same manner as the simple portal with hinged posts in Fig. 100.

For the calculation of the stresses in a portal, see Problem 22, Chapter IX.

CHAPTER VIII.

STRESSES IN PINS, ECCENTRIC AND COMBINED STRESSES, DEFLECTION OF TRUSSES, STRESSES IN ROLLERS, AND CAMBER.

STRESSES IN PINS.—A pin under ordinary conditions is a short beam and must be designed (1) for bending, (2) for shear, and (3) for bearing. If a pin becomes bent the distribution of the loads and the calculation of the stresses are very uncertain.

The cross-bending stress, S, is found by means of the fundamental formula for flexure, $S = Mc/I$, where the maximum bending moment, M, is found as explained later; I is the moment of inertia; and c is one-half the radius of a solid or hollow pin.

The safe shearing stresses given in standard specifications are for a uniform distribution of the shear over the entire cross-section, and the actual unit shearing stress to be used in designing will be equal to the maximum shear divided by the area of the cross-section of the pin.

The bearing stress is found by dividing the stress in the member by the bearing area of the pin, found by multiplying the thickness of the bearing plates by the diameter of the pin.

Calculation of Stresses.—The method of calculation will be illustrated by calculating the stresses in the pin at U_1 in (a) Fig. 103. In the complete investigation of the pin U_1, it would be necessary to calculate the stresses when the stress in U_1U_2 was a maximum, and when the stress in U_1L_2 was a maximum. Only the case where the stress in U_1U_2 is a maximum will be considered. However, maximum stresses in pins sometimes occur when the stress in U_1L_2 is a maximum, and this case should be considered in practice.

Bending Moment.—The stresses in the members are shown in (c) Fig. 103, which gives the force polygon for the forces. The makeup of the members is shown in (a), and the pin packing on one side is

shown in (b). The stresses shown in (c) are applied one-half on each side of the member, the pin acting like a simple beam. The stresses are assumed as applied at the centers of the members.

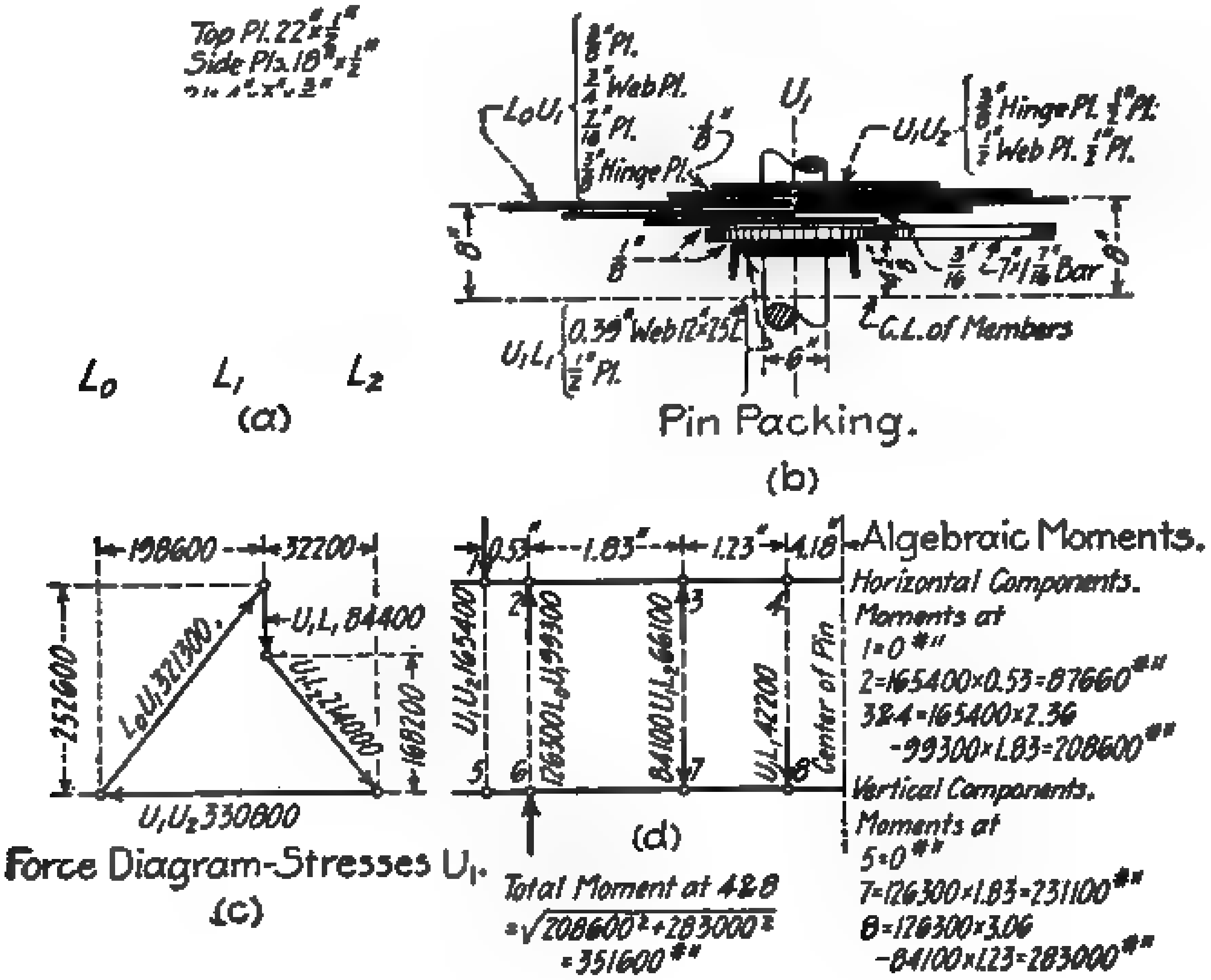


FIG. 103. STRESSES IN A PIN; ALGEBRAIC SOLUTION.

Algebraic Method.—The amounts of the forces and the distances between their points of application as calculated from (b) are shown in (d) Fig. 103. The horizontal and vertical components of the forces are considered separately, the maximum horizontal bending moment and the maximum vertical bending moment are calculated for the same point, and the resultant moment is then found by means of the force triangle.

In (d) the horizontal bending moments are calculated about the points 1, 2, 3, 4; the maximum horizontal moment is to the right of 3, and is 208,600 in.-lbs. The vertical bending moments are calculated

about points 5, 6, 7, 8; the maximum vertical bending moment is to the right of 8, and is 283,000 in.-lbs. The maximum bending moment is at and to the right of 4 and 8, and is $\sqrt{208{,}600^2 + 283{,}000^2} = 351{,}600$ in.-lbs. Substituting in the formula $S = Mc/I$, the maximum bending

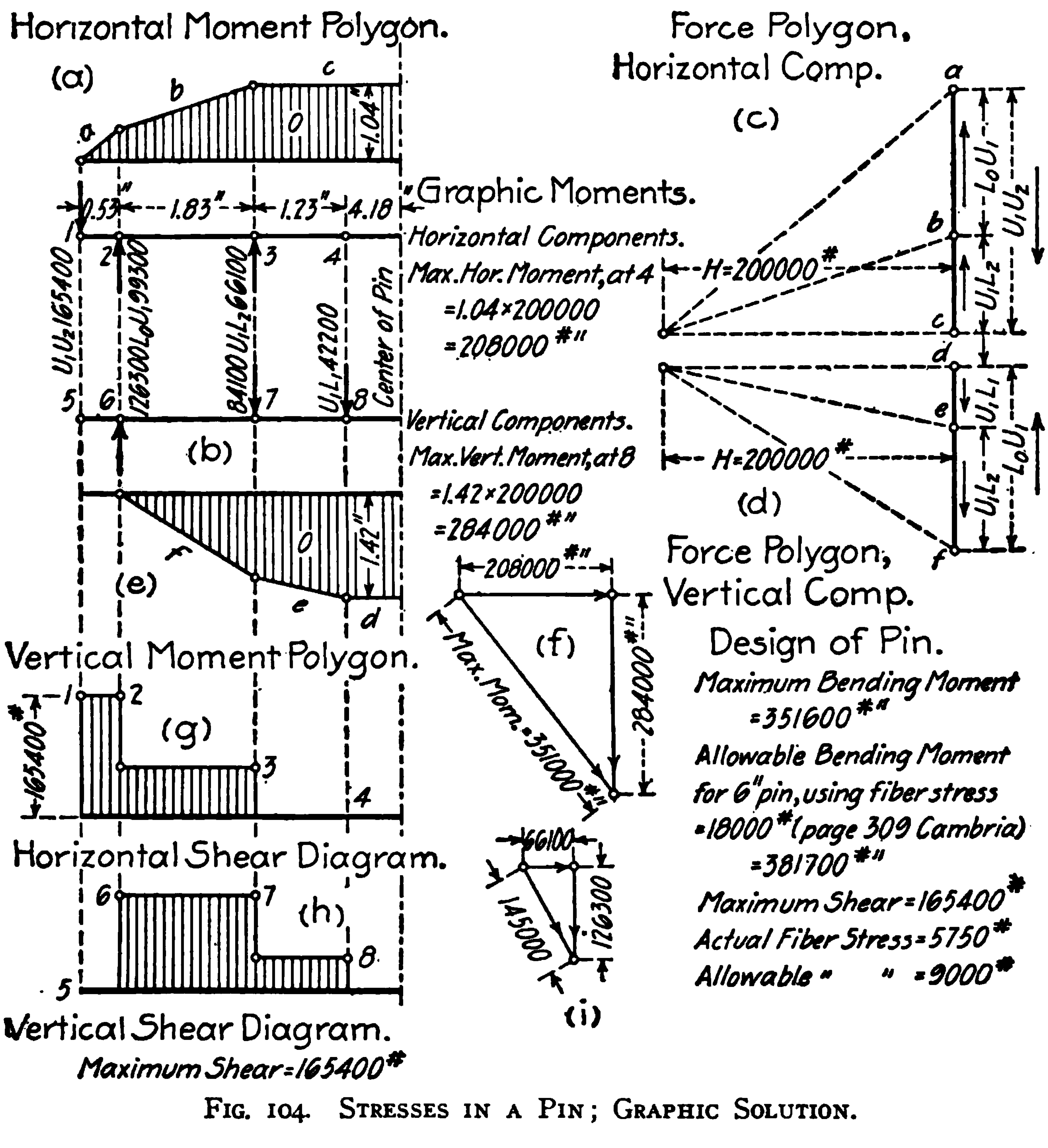


FIG. 104. STRESSES IN A PIN; GRAPHIC SOLUTION.

stress is $S = 16{,}600$ lbs. The allowable bending stress for which this bridge was designed was 18,000 lbs. per square inch.

Graphic Method.—The amounts of the forces and the distances between their points of application are shown in (b) Fig. 104. The force polygon for the horizontal components is given in (c), and the

bending moment polygon is given in (a). The maximum horizontal bending moment will be to the right of 3, and will be $H \times y = 200,000 \times 1.04 = 208,000$ in.-lbs. The force polygon for the vertical forces is given in (d) and the bending moment polygon is given in (e). The maximum vertical bending moment is to the right of 8, and is $H \times y = 200,000 \times 1.42 = 284,000$ in.-lbs. The maximum bending moment will occur at and to the right of 4 and 8, and will be 351,000 in.-lbs., as shown in (f).

Shear.—The shear is found for both the horizontal and vertical components as in a simple beam, and is equal to the summation of all the forces to the left of the section. The horizontal shear diagram is shown in (g), and the vertical shear diagram is shown in (h) Fig. 104. The maximum horizontal shear is between 1 and 2, and is 165,400 lbs. The shear between 2 and 3 is $165,400 - 99,300 = 66,100$ lbs. The maximum vertical shear is between 6 and 7, and is 126,300 lbs. The resultant shear between 2 and 3, and 6 and 7, is $\sqrt{126,300^2 + 66,100^2} = 145,000$ lbs. as in (i), which is less than the horizontal shear between 1 and 2. The maximum shear, therefore, comes between 1 and 2, and is 165,400 lbs. The maximum shearing unit stress is 5,750 lbs. The allowable shearing stress was 9,000 lbs.

Bearing.—The bearing stress in L_0U_1 is $160,650 \div 6 \times 1.94 = 13,800$ lbs. Bearing stress in U_1U_2 is $165,400 \div 6 \times 1.88 = 14,600$ lbs. Bearing stress in U_1L_1 is $42,200 \div 6 \times 0.89 = 7,900$ lbs. Bearing stress in U_1L_2 is $107,000 \div 6 \times 1\frac{7}{16} = 12,400$ lbs. The allowable bearing stress was 15,000 lbs.

For the calculation of the stresses in the pins of a 160-ft. steel highway bridge, see Chapter XXII, Part III.

COMBINED AND ECCENTRIC STRESSES.—The combined stress due to direct and cross-bending in a tie or strut is given by the formula*

$$f = f_2 \pm f_1 = \frac{P}{A} \pm \frac{M_1 \cdot y_1}{I \pm \dfrac{P \cdot l^2}{c \cdot E}} \tag{63}$$

* For the derivation of this formula, see "Steel Mill Buildings," Chapter XV.

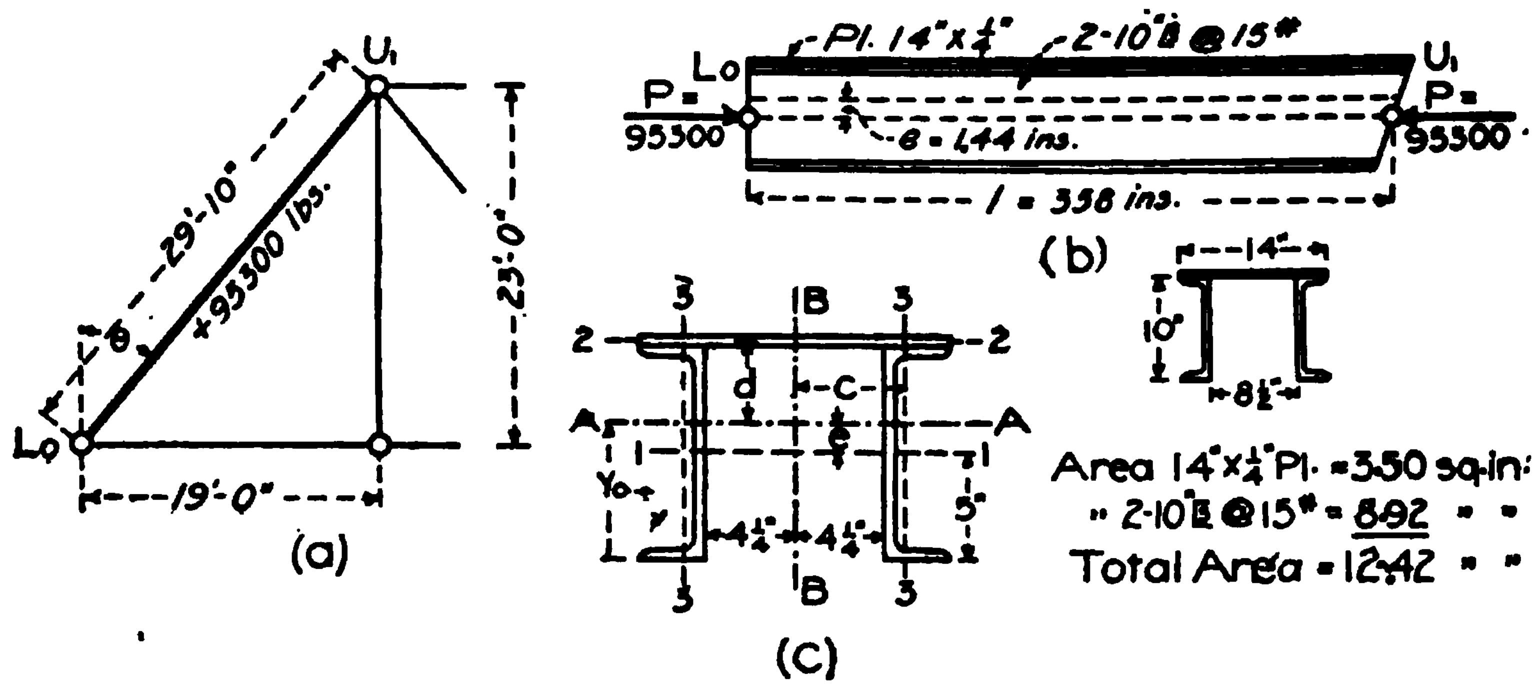
where P = total direct stress in the member in lbs.;
l = length of the member in ins.;
I = moment of inertia of the member in ins. to the fourth power;
y_1 = distance in ins. from the neutral axis to the most remote fiber on the side for which the stress is desired;
E = modulus of elasticity of the material in lbs. per sq. in.;
A = area of the member in sq. ins.;
f_1 = fiber stress due to cross-bending;
$f_2 = P/A$ = direct unit stress;
M_1 = bending moment on the section in in.-lbs.;
c = a coefficient depending upon the method of loading and the condition of the ends, and is usually taken as 10 for struts with hinged ends, 24 for struts with one end hinged and the other end fixed, and 32 for both ends fixed.

The plus sign in the denominator of (63) is to be used when P is a tensile stress, and the minus sign is to be used when P is a compressive stress. If the member is inclined at an angle θ to the vertical, the stress f_1 should be multiplied by $\sin\theta$. For an eccentric stress the bending moment is $M_1 = P \cdot e$, where P is the total direct stress in the member and e is the eccentricity of the load in ins. (distance from the line of action of the force to the neutral axis of the member).

Combined Compression and Cross-bending.—The method of calculating direct and cross-bending stresses will be illustrated by calculating the stresses in the end-post of a bridge, Fig. 105, due to direct compression, weight, eccentricity of loading, and wind moment.

The end-post is composed of two 10-inch channels weighing 15 lbs. per foot with a 14″ × ¼″ plate riveted on the upper side, and laced on the lower side with single lacing. The pins are placed in the center of the channels, giving an eccentricity of $e = 1.44$ inches. The compressive stress, P, produces a uniform compression on all fibers of the section; weight of the member causes tension on the lower and compression on the upper fibers; eccentricity of the load P causes compression on lower and tension on upper fibers; and wind moment causes compression on the windward and tension on the leeward fibers. The

maximum fiber stress will come at the foot of the portal knee brace either on the upper or lower fibers on the windward side of the post, depending upon whether the stress due to weight is greater than the stress due to eccentric loading, or the reverse.



To locate neutral axis A-A take moments about lower edge of channels

$$Y_0 = \frac{8.92 \times 5 + 3.5 \times 10.125}{12.42} = 6.44''$$

Eccentricity, $e = 6.44 - 5.00 = 1.44''$

Moment of Inertia, I_A, about A-A	Moment of Inertia, I_B, about B-B
Let $I_{[}$ = I of [about axis 1-1 = 133.8	Let $I_{[}$ = I of [about axis 3-3 = 4.6
$I_{pl.}$ = I of Pl. about axis 2-2 = .02	$I_{pl.}$ = I of Pl. about axis B-B = 57.17
$A_{[}$ = Area of [= 8.92 sq. in	$A_{[}$ = Area of [= 8.92 sq. in.
$A_{pl.}$ = Area of Pl. = 3.50 sq. in.	$A_{pl.}$ = Area of Pl. = 3.50 sq. in.
Then $I_A = I_{[} + A_{[}e^2 + I_{pl.} + A_{pl.}d^2$	Then $I_B = I_{[} + A_{[}(4.25+.64)^2 + I_{pl.}$
$= 133.8 + 8.92 \times (1.44)^2 + .02 + 3.5(3.685)^2$	$= 4.6 + 8.92(4.89)^2 + 57.17$
$= 199.8$	$= 275.0$
Radius of gyration, $r_A = \sqrt{\frac{199.8}{12.42}} = 4.0''$	Radius of gyration, $r_B = \sqrt{\frac{275.0}{12.42}} = 4.7''$

FIG. 105.

Stress Due to Weight of Member.—The total weight of the member is as follows:

Two 10" [s @ 15 lbs., 30' 0" long	=	900 lbs.
One 14" × ¼" Pl. @, 11.9 lbs., 30' 0" long	=	357 lbs.
Details and lacing, 26 per cent	=	328 lbs.
Total weight, *W*,	=	1,585 lbs.

Bending moment due to weight of the member, $M = \frac{1}{8}W \cdot l \cdot \sin\theta$.

Stress due to weight

$$f_w = \frac{M \cdot y_1}{I - \frac{P \cdot l^2}{10E}} = \frac{\frac{1}{8} W \cdot l \cdot \sin\theta \cdot y_1}{I - \frac{P \cdot l^2}{10E}}$$

Stress due to weight in upper fiber

$$f_w = \frac{\frac{1}{8} \times 1{,}585 \times 358 \times .633 \times 3.81}{199.8 - \frac{95{,}300 \times 358^2}{10 \times 28{,}000{,}000}} = +\ 1{,}100 \text{ lbs. (compression)}$$

Stress due to weight in lower fiber

$$f_w' = 6.44/3.81 \times 1{,}100 = -\ 1{,}860 \text{ lbs. (tension)}$$

Stress Due to Eccentric Loading.—The stress in the extreme fiber due to eccentric loading will be

$$f_e = \frac{M \cdot y_1}{I - \frac{P \cdot l^2}{10E}} = \frac{P \cdot e \cdot y_1}{I - \frac{P \cdot l^2}{10E}}$$

Eccentric stress in upper fiber

$$f_e = \frac{95{,}300 \times 1.44 \times 3.81}{199.8 - \frac{95{,}300 \times 358^2}{10 \times 28{,}000{,}000}} = -\ 3{,}347 \text{ lbs. (tension)}$$

Eccentric stress in lower fiber

$$f_e' = 6.44/3.81 \times 3{,}347 = +\ 5{,}657 \text{ lbs. (compression)}$$

The resultant stress due to eccentric loading and weight will be

$$\begin{aligned} f_1 &= f_e + f_w \\ &= -\ 3{,}347 + 1{,}100 = -\ 2{,}247 \text{ lbs. in upper fiber} \\ &= +\ 5{,}657 - 1{,}860 = +\ 3{,}797 \text{ lbs. in lower fiber} \end{aligned}$$

The maximum stress in the member due to direct loading, weight of member and eccentric loading will occur in the lower fiber and will be

$$\begin{aligned} f_2 + f_1 &= P/A + f_e + f_w \\ &= 95{,}300/12.42 + 3{,}797 = +\ 11{,}470 \text{ lbs.} \end{aligned}$$

To determine the position of the pin so that stress due to weight will neutralize stress due to eccentric loading make

$$P \cdot e' = \tfrac{1}{8} W \cdot l \cdot \sin \theta,$$

where e' is the distance of the pin below the neutral axis.

Substituting and solving, $95{,}300 \times e' = \tfrac{1}{8}(1585 \times 358 \times .633)$

$$e' = .48''$$

Stress Due to Wind Moment.—Before calculating the stress due to wind moment, it will be necessary to determine whether the end-post is fixed or hinged.

If the end-post, Fig. 106, is fixed, the negative moment developed at the lower pin, L_0, will be $M = H \cdot d/2 = (3{,}200 \times 226)/2 = 361{,}600$ in.-lbs.

In order to obtain this condition of fixidity, the stress in the member must develop a resisting moment equal in amount.

Therefore the post may be considered fixed if

$$\tfrac{1}{2}(95{,}300 - V)a \gtreqless \tfrac{1}{2} H \cdot d$$

or

$$\tfrac{1}{2}(95{,}300 - 8{,}520)8 \gtreqless 361{,}600$$

but

$$347{,}120 < 361{,}600$$

and the end-post will not be fixed.

(While this is the usual solution, the resisting moment certainly reduces the bending moment and the bending stress is less than computed below. See solution in Fig. 290, Chapter XXII.)

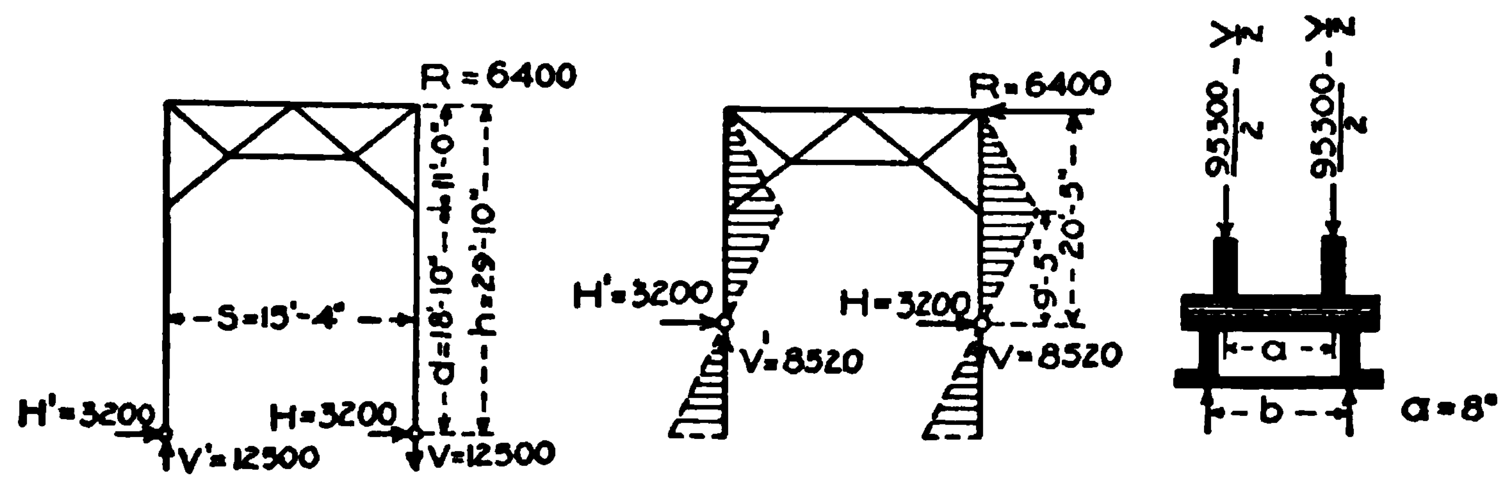
The maximum moment will then occur at the foot of the portal knee brace and will be $M = 3{,}200 \times 226 = 723{,}200$ in.-lbs.

Stress due to wind moment is a maximum in the leeward post, and is

$$f_w = \frac{M \cdot y_1}{I - \dfrac{P \cdot l^2}{10E}} = \frac{723{,}200 \times 7}{275 - \dfrac{(95{,}300 + 12{,}500)358^2}{10 \times 28{,}000{,}000}}$$

$$f_w = \pm\, 22{,}480 \text{ lbs.}$$

Stress f_w, is compression on the windward side and tension on the leeward side of the member.



Portal, Columns Hinged. (a)
Portal, Columns Fixed. (b)
Windward Pedestal. (c)

FIG. 106.

In this case the maximum stress comes on the lower fibers of the windward side of the post.

Combined Tension and Cross-bending.—The stress due to cross-bending when the member is also subjected to direct tension is given by the formula

$$f_2 = \frac{M_1 \cdot y_1}{I + \frac{P \cdot l^2}{c \cdot E}} \tag{64}$$

the nomenclature being the same as in (63). The constant c is taken equal to 10 where the ends are hinged.

Stress in a Bar Due to its Own Weight.—Let b = breadth of bar in inches; h = depth of bar in inches; w = weight of bar per lineal inch = 0.28 $b \cdot h$ lbs.; $f_2 = P/b \cdot h$ = direct unit stress in lbs. per sq. in.

We will also have $y_1 = \frac{1}{2}h$; $M_1 = \frac{1}{8}w \cdot l^2$; $P = f_2 \cdot b \cdot h$.

Substituting in (64), we have

$$f_1 = \frac{\frac{1}{8}w \cdot l^2 \cdot \frac{1}{2}h}{\frac{b \cdot h^3}{12} + \frac{f_2 \cdot b \cdot h \cdot l^2}{10 \times 28{,}000{,}000}} = \frac{4{,}900{,}000h}{f_2 + 23{,}000{,}000\left(\frac{h}{l}\right)^2} \tag{65}$$

where f_1 is the extreme fiber stress in the bar due to weight, and is tension in lower fiber and compression in upper fiber.

If the bar is inclined, the stress obtained by formula (65) must be multiplied by the sine of the angle that the bar makes with a vertical line. Formula (65) is much more convenient for actual use than formula (64).

Diagram for Stress in Bars Due to Their Own Weight.—Taking the reciprocal of (65), we have

$$\frac{1}{f_1}=\frac{f_2}{4{,}900{,}000h}+\frac{23{,}000{,}000\left(\frac{h}{l}\right)^2}{4{,}900{,}000h}=y_1+y_2$$

and

$$f_1=1/(y_1+y_2) \tag{66}$$

Fig. 107 gives values of y_1 for different values of f_2, and values of y_2 for different values of the length in feet, L. The values of y_1 and y_2 can be read off the diagram directly for any value of h, f_2 and L. And then, if the sum of y_1 and y_2 be taken on the lower part of the diagram, the reciprocal, which is the fiber stress f_1, may be read off the right hand side.

The use of the diagram will be illustrated by two problems:

PROBLEM 1.—Required the stress in a 4″ × 1″ eye-bar, 20′ 0″ long, which has a direct tension of 56,000 lbs.

In this case, $h=4''$, $L=20'\ 0''$, and $f_2=14{,}000$ lbs. per sq. in. The stress due to weight, f_1, is found as follows: On the bottom of the diagram, Fig. 107, find $h=4$ inches, follow up the vertical line to its intersection with inclined line marked, $L=20$ feet, and then follow the horizontal line passing through the point of intersection out to the left margin and find, $y_2=3.3$ tens of thousandths; then follow the vertical line, $h=4$ inches, up to its intersection with inclined line marked, $f_2=14{,}000$, and then follow the horizontal line passing through the point of intersection out to the left margin and find, $y_1=7.2$ tens of thousandths.

Now to find the reciprocal of $y_1+y_2=7.2+3.3=10.5$, find value of $y_1+y_2=10.5$ on lower edge of diagram, follow vertical line to its

intersection with inclined line marked "Line of Reciprocals" and find stress f_1 by following horizontal line to right hand margin to be

$$f_1 = 950 \text{ lbs. per sq. in.}$$

By substituting in (65) and solving we get $f_1 = 960$ lbs. per sq. in.

PROBLEM 2.—Required the stress in a 5″ × ¾″ eye-bar, 30′ 0″ long, which has a direct tension of 60,000 lbs., and is inclined so that it makes an angle of 45° with a vertical line.

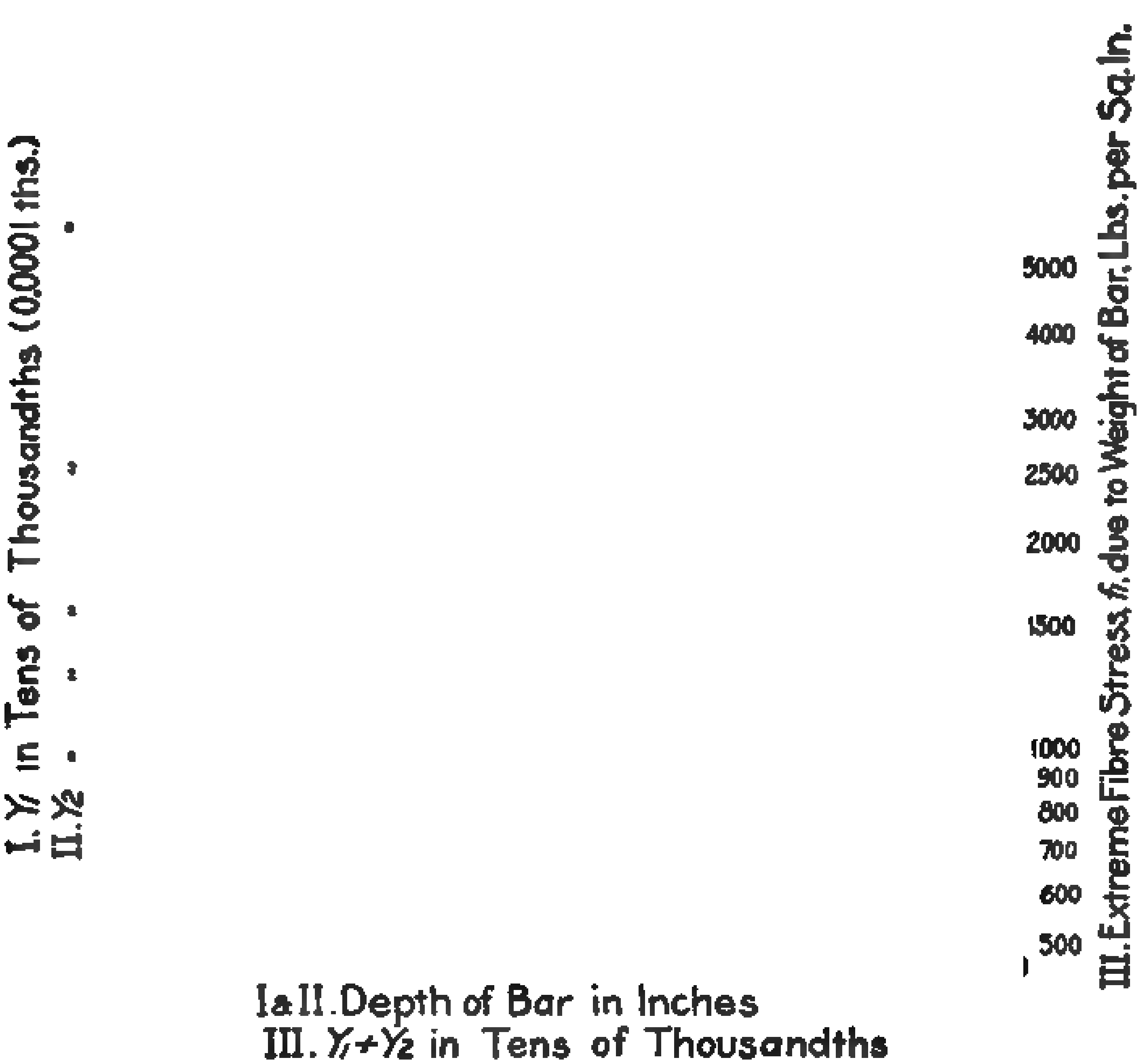


FIG. 107. DIAGRAM FOR FINDING STRESS IN BARS DUE TO THEIR OWN WEIGHT.

In this case, $h = 5''$, $L = 30'\ 0''$, $f_2 = 16{,}000$ lbs., and $\theta = 45°$. From the diagram, Fig. 107, as in Problem 1, $y_2 = 1.8$ tens of thousandths, and $y_1 = 6.5$ tens of thousandths, and

$$f_1 = 1/(y_1 + y_2) \times \sin\theta = 1{,}200 \times \sin\theta$$
$$= 850 \text{ lbs. per sq. in.}$$

Relations Between h, f_1, f_2 and L.—For any values of f_2 and L, f_1 will be a maximum for that value of h which will make $y_1 + y_2$ a minimum. This value of h will now be determined. Differentiating equation (65) with reference to f_1 and h, we have after solving for h after placing the first derivative equal to zero

$$h = l\sqrt{f_2}/4{,}800 \qquad (67)$$

in which h is the depth of bar which will have a maximum fiber stress for any given values of l and f_2.

Now if we substitute the value of h in (67) back in equation (65), we find that f_1 will be a maximum when $y_1 = y_2$.

Now in the diagram the values of y_1 and y_2 for any given values of f_2 and L will be equal for the depth of bar, h, corresponding to the intersection of the f_2 and L lines.

It is therefore seen that every intersection of the inclined f_2 and L lines in the diagram, has for an abscissa a value h, which will have a maximum fiber stress f_1, for the given values of f_2 and L.

For example, for $L = 30$ feet and $f_2 = 12{,}000$ lbs., we find $h = 8.3$ inches and $f_1 = 1{,}700$ lbs. For the given length L and direct fiber stress f_2, a bar deeper or shallower than 8.3 inches will give a smaller value of f_1 than 1,700 lbs.

STRESSES IN AN ECCENTRIC RIVETED CONNECTION.—In Fig. 108 the riveted connection carries a stress of $P = 10{,}000$ lbs. The four rivets transmit a direct shear of 10,000 lbs. or 2,500 lbs. each, and a bending moment of $10{,}000 \times 4\frac{1}{2} = 45{,}000$ in.-lbs. The shear that resists moment in each rivet acts with an arm of 2.8 ins. If R is the shear in each rivet due to moment, $4R \times 2.8 = 45{,}000$ in.-lbs., and $R = 4{,}018$ lbs.

The total shear on rivet 2, is $4,018 - 2,500 = 1,518$ lbs.; on rivet 3, is $4,018 + 2,500 = 6,518$ lbs.; and on rivets 1 and $4 = \sqrt{2,500^2 + 4,018^2}$ $= 4,740$ lbs.

If the rivets are located at unequal distances from the center of gravity of the rivets, let a represent the shear on a rivet at a unit's distance from the center of gravity; then the shear on a rivet at a distance d_1 from the center of gravity will be $a \cdot d_1$, and the resisting moment will be $a \cdot d_1^2$. The shear on a rivet at a distance d_2 from the center of gravity will be $a \cdot d_2$, and the resisting moment will be $a \cdot d_2^2$. The total resisting moment of the connection will then be $\Sigma a \cdot d^2 = M$.

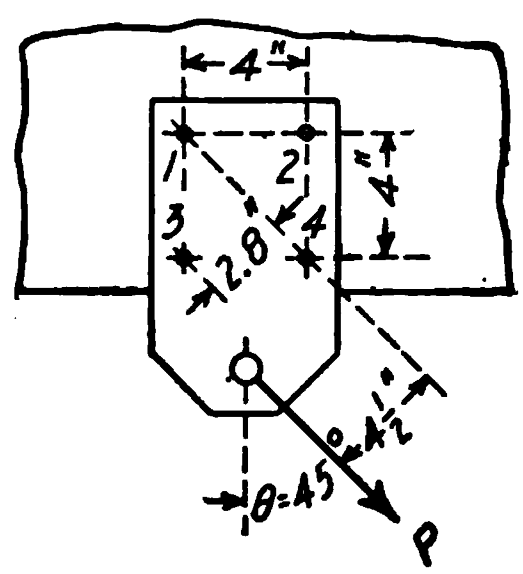


FIG. 108.

For the calculation of the stresses in the rivets of a standard riveted connection see the author's "Steel Mill Buildings," Chapter XV.

DEFLECTION OF TRUSSES.—When the members of a truss are stressed, the lengths of the members in compression are decreased in length, while the members in tension are increased in length. These changes in the lengths of the members cause the upper and lower chord panel points to deflect, while the positions of all other points are changed. If the left end of a bridge truss is fixed the right end will move if it is resting on free rollers. To calculate the movement of the right end of the truss proceed as follows:

In Fig. 109 the truss is fixed at L_0, and is free to move at L_0', and is loaded with a load W. Under the action of the load, L_0' will move a distance Δ. Now assume that all the members are rigid with the exception of 1–3', which is increased in length the distance δ, under

the action of the external load W. The movement of the joint L_0' will be Δ', and will be due to the change in length δ, of the member 1–y. Let H be the horizontal reaction necessary to bring L_0' back to its original position, and let $U \cdot H$ be the stress in the member 1–y

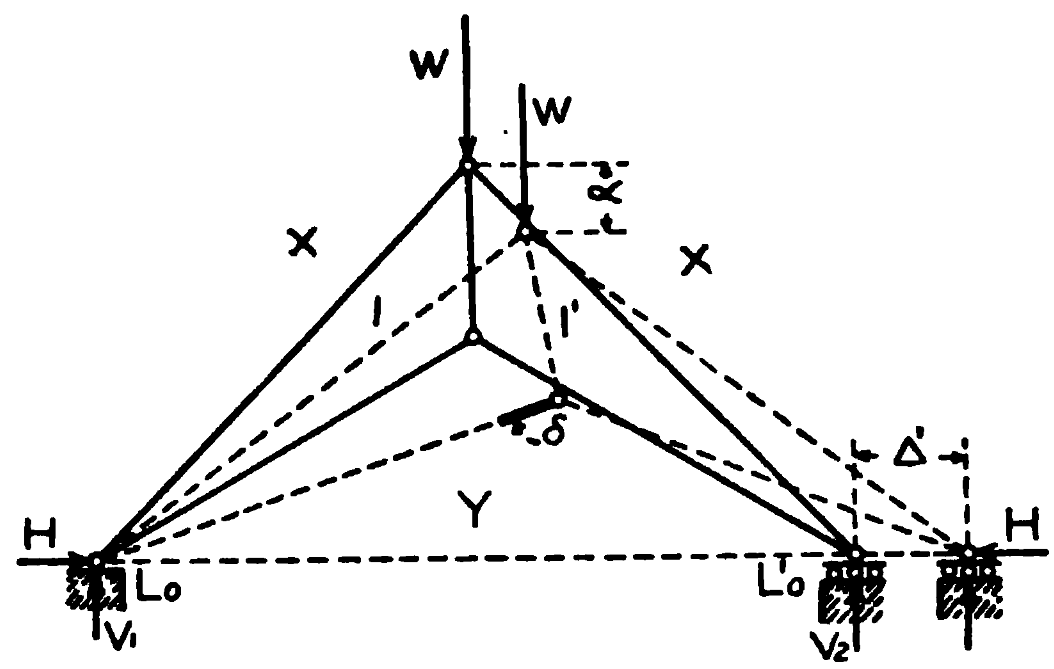


FIG. 109.

due to the horizontal thrust H. Now the internal work $\frac{1}{2}\delta \cdot H \cdot U$ in shortening the member 1–y to its original length will be equal to the external work $\frac{1}{2}H \cdot \Delta'$, required to bring the hinge L_0' back to its original position, and

$$\tfrac{1}{2}H \cdot \Delta' = \tfrac{1}{2}\delta \cdot H \cdot U$$

and

$$\Delta' = \delta \cdot U \tag{68}$$

but $\delta = P \cdot L/E$, where P is the unit stress in the member 1–y due to the load W; L is the length of the member 1–y in the same units as Δ'; and E is the modulus of elasticity of the material of the member in lbs. per sq. in. Substituting this value of δ in (68) we have

$$\Delta' = P \cdot U \cdot L/E \tag{69}$$

where U is the stress in the member due to a load unity at L_0' acting in the line in which Δ' is measured.

Now if each one of the remaining members of the truss is assumed as distorted in turn, the other members meanwhile remaining rigid, the distortion at L_0' will be represented by the general equation (69), and the total deformation, Δ, at L_0' will be

$$\Delta = \Sigma(P \cdot U \cdot L/E) \tag{70}$$

Algebraic Solution.—It is required to calculate the deflection of the panel point L_3 in the lower chord of the 160-ft. span Pratt highway truss in Fig. 111, the stresses in the members, the areas of the members, and the lengths of the members being given in Table XI. The stresses in column 4 in Table XI are calculated for a full dead load and live load on the truss. In column 5 values of unit stress, P, are given, in column 6 values of $P \cdot L/E$ are given for $E = 30{,}000{,}000$ lbs. per sq. in. The values of U in column 8 were calculated by placing a load of 1 lb. at L_3. The values of $P \cdot U \cdot L/E$ are given in column 9, and $\Sigma(P \cdot U \cdot L/E)$ is 1.05 ins To calculate the deflection at any other point, new values of U must be calculated for a force of 1 lb. acting at the point at which the deflection is to be calculated, and acting in the direction that the deflection is to be measured.

Graphic Solution.—The principle upon which the construction of the deformation diagram is based is as follows: Take the two members

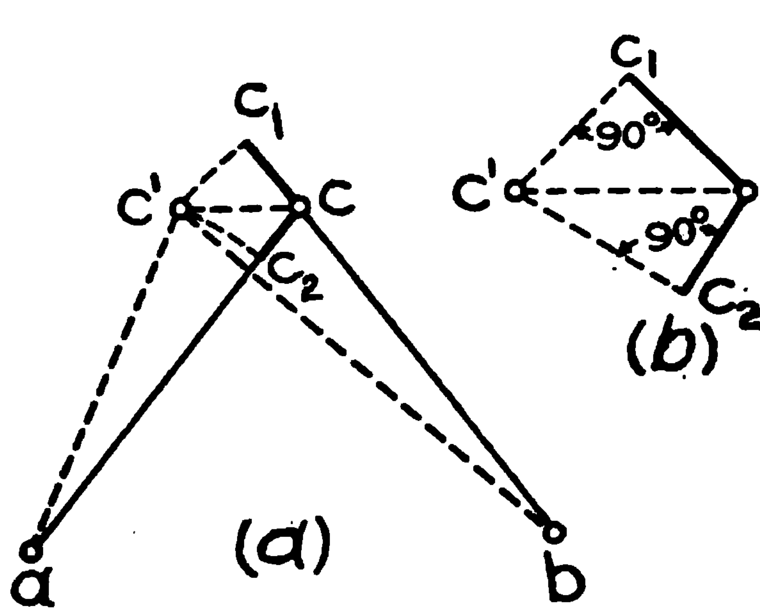
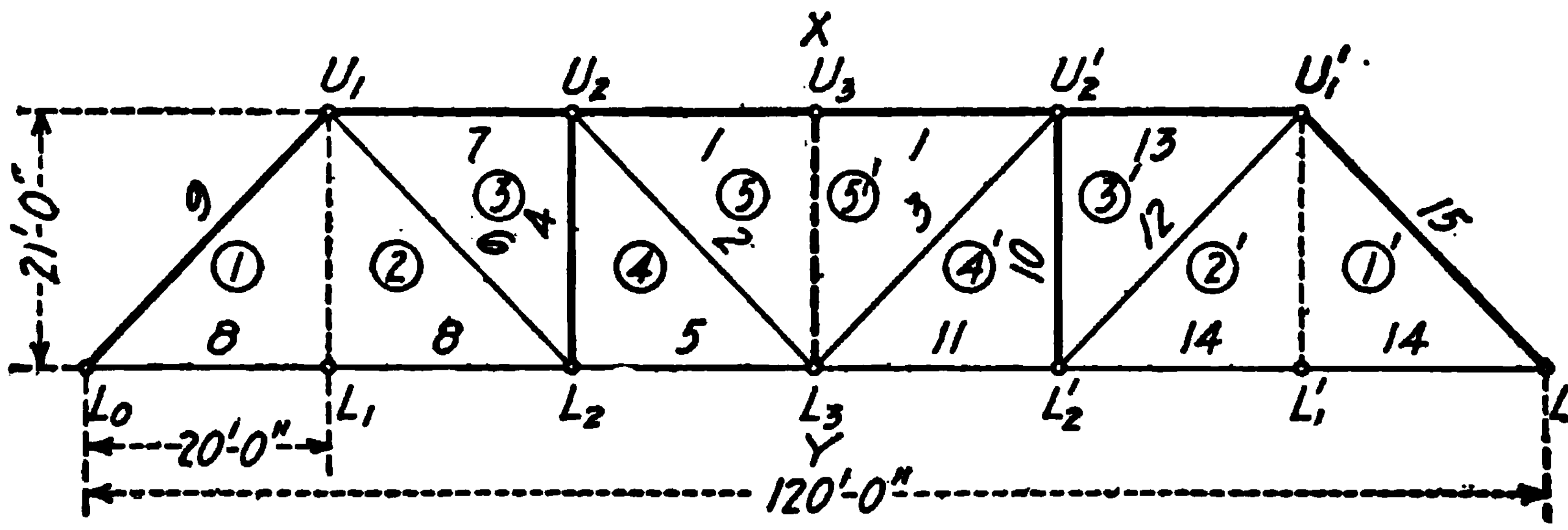


FIG. 110.

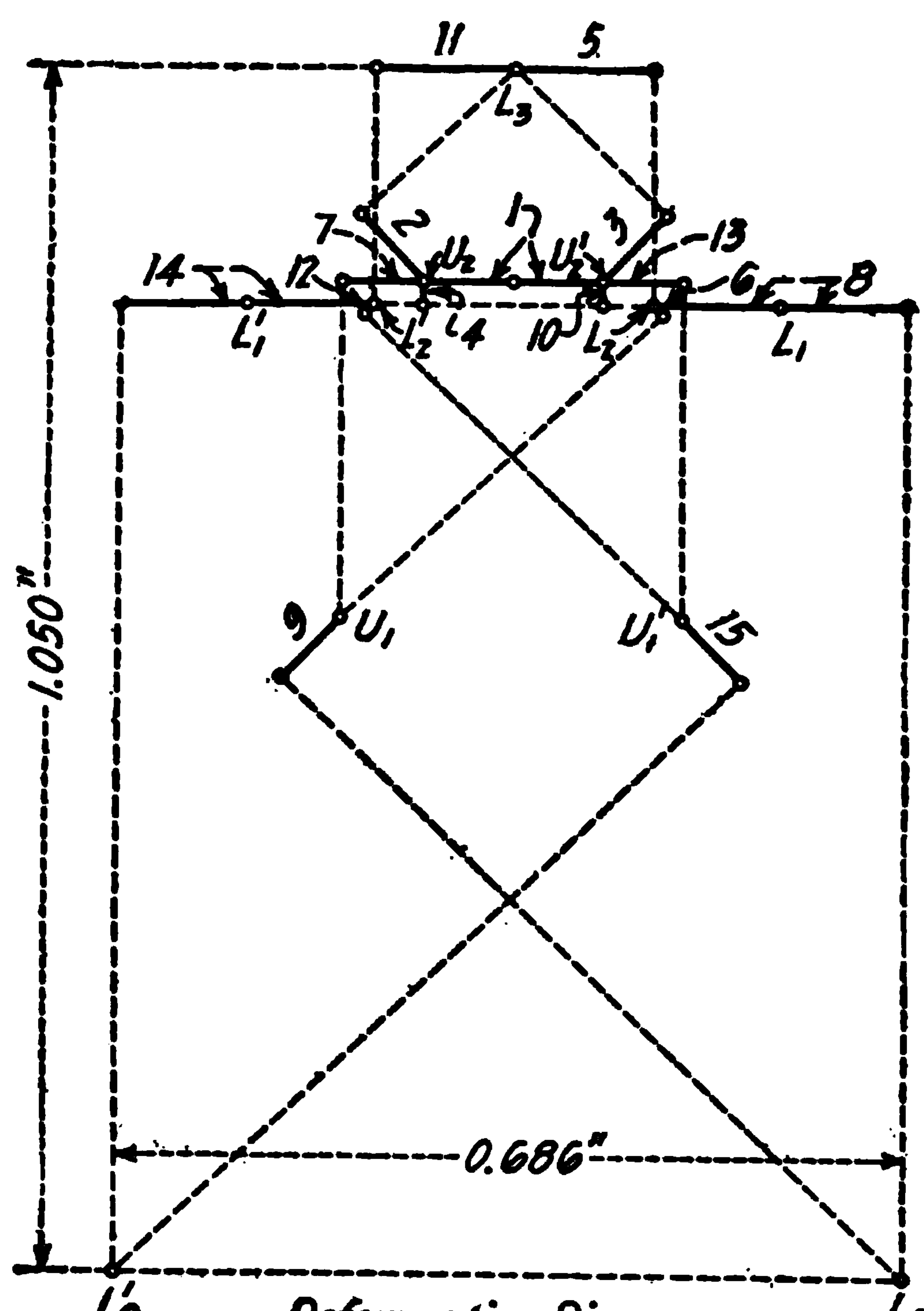
a–c and b–c in (a) Fig. 110 meeting at the point c. Assume that a–c is shortened to a–c_2 and b–c is lengthened to b–c_1 as shown, it is required to calculate the new position, c', of the point c. With center at a and radius a–c_2 describe arc c'–c_2; the point c' comes at some point on this line. With center b and radius b–c_1 describe arc c'–c_1 cutting arc c'–c_2 at c', which is the point desired. Since the deformations are always small as compared with the lengths of the members, the arcs may be replaced by perpendiculars as shown in (b) Fig. 110, and the members themselves need not be drawn.

To draw the deformation diagram for the truss in Fig. 111, proceed as follows: First calculate Table XI as for the algebraic method, column 7 giving the order in which the members will be used in the



Truss Diagram.

FIG. 111.



Deformation Diagram.

FIG. 112.

deformation diagram. Now begin with the member marked 1, and lay off in Fig. 112 the deformation of the two members marked $1 = +2 \times 0.078 = +0.156$ ins. parallel to the member $U_2U_3U_2'$ to the prescribed scale, and mark the left end U_2 and the right end U_2'. Now lay off the deformation $2 = -0.08$ ins. from U_2 and toward the joint U_2, and parallel to member 2; and lay off deformation $3 = -0.08$ ins. from U_2' and toward the joint U_2', and parallel to member 3. Perpendiculars erected at the ends of deformations 2 and 3 will meet in the new position of L_3. The vertical distance between U_2 and L_3 in the diagram will be the difference in deflection of the points U_2 and L_3. At U_2 in the diagram lay off deformation $4 = +0.02$ ins. away from the joint U_2 and parallel to member 4, and at L_3 lay off deformation $5 = -0.121$ ins. toward the joint L_3 and parallel to member 5. Then perpendiculars erected at the ends of deformations 4 and 5 will meet in the new position of L_2. In like manner perpendiculars erected at the ends of deformations 6 and 7 give the new position of point U_1 in the diagram, and finally perpendiculars erected at the ends of deformations 9 and 2×8 will give the new position of the point L_0. The deformation diagram for the right half of the truss is constructed in the same manner. The increase in the length of the span is 0.686 ins., while the deflection of point L_3 below the abutments is 1.05 ins., as was calculated algebraically.

TABLE XI.

ALGEBRAIC CALCULATION OF DEFORMATIONS.

Member.	Area in Sq. In.	Length L in In.	Stress in Lbs.	Unit Stress P in Lbs.	$\frac{PL}{E}$	No. Mem.	U	$\frac{PUL}{E}$
X-3	8.70	240	75,600	+ 8,700	+ 0.070	7	+ 0.96	+ 0.067
X-5	8.70	240	85,100	+ 9,800	+ 0.078	1	+ 1.44	+ 0.112
Y-1	3.44	240	47,300	− 13,800	− 0.110	8	− 0.48	+ 0.053
Y-2	3.44	240	47,300	− 13,800	− 0.110	8	− 0.48	+ 0.053
Y-4	5.00	240	75,600	− 15,100	− 0.121	5	− 0.96	+ 0.116
X-1	10.70	348	68,650	+ 6,400	+ 0.074	9	+ 0.69	+ 0.051
1-2	1.44	252	19,900	− 1,400	− 0.012		0.0	0.0
2-3	3.13	348	31,200	− 1,000	− 0.012	6	− 0.69	+ 0.008
3-4	3.90	252	9,450	+ 2,400	+ 0.020	4	+ 0.50	+ 0.010
4-5	2.00	348	13,700	− 6,900	− 0.080	2	− 0.69	+ 0.055
5-5′	3.90	252	0	0	0.0		0.0	0.0

Total deformation $= +0.525 \times 2 = 1.050$ ins. + 0.525

The graphic method gives the relative positions of all points in the truss, while the algebraic method gives the deflection of one point, only.

For the deformation diagrams of trusses unsymmetrically loaded, and for the methods of calculating the stresses in two-hinged arches see the author's " Steel Mill Buildings," Chapter XIV.

STRESSES IN ROLLERS.—When a cylindrical roller is pressed between two plates the roller is deformed so that the linear element of the roller in contact is spread out as the pressure increases. It has been found by experiment that the plates are but little deformed in comparison with the deformation of the rollers for stresses within the elastic limit, so that the entire deformation may be considered as occurring in the rollers. (For a more complete discussion see Merriman's " Mechanics of Materials.")

In (b) Fig. 113 the vertical diameter A–A is shortened to B–B, and the shortening in a half diameter is $A\text{–}B = e$; also let y be the

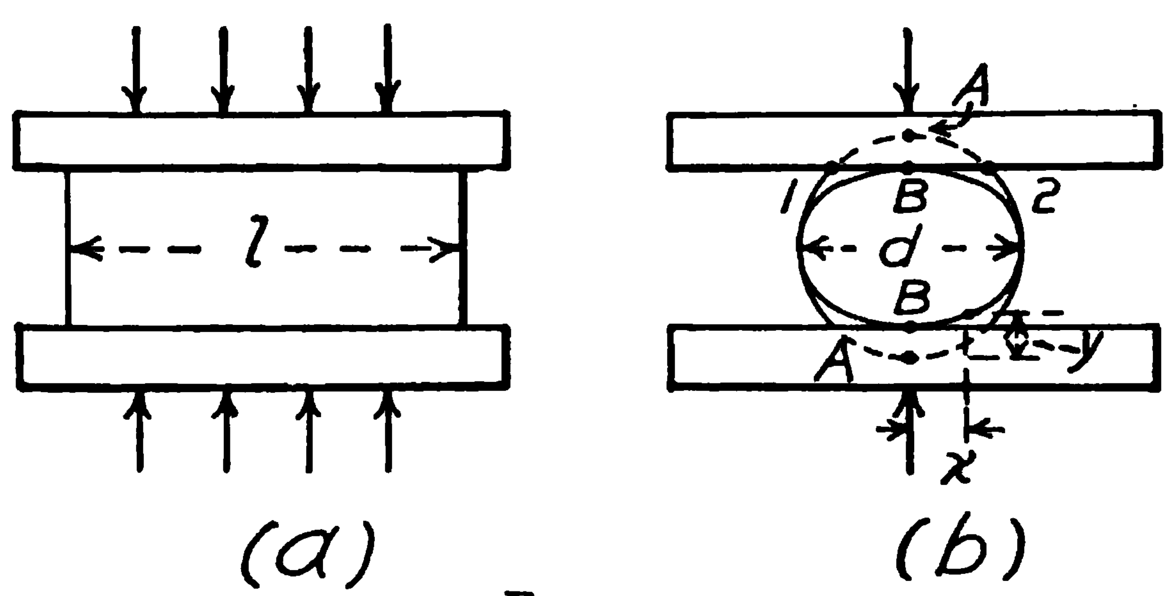


FIG. 113.

shortening in any half chord. Now if the stress at B is S, and at the point whose coördinates are x, y is S', then if the elastic limit of the material is not exceeded

$$S/S' = e/y, \text{ or } S' = S \cdot y/e$$

Now $e/2d$ is the unit shortening in the vertical diameter, and this is equal to S/E, and

$$S/E = e/2d \tag{71}$$

Now the stress S' acts over the entire area $l \cdot dx$, and

$$\int S' \cdot l \cdot dx = W \tag{72}$$

where W is the total stress on the rollers. Equations (71) and (72) are the equations for finding S and e. Substituting $S' = S \cdot y/e$ in (72) we have

$$S \cdot l \int y \cdot dx = W \cdot e \tag{73}$$

Now $\int y \cdot dx =$ the area of the segment compressed, which may be considered a parabola $= \frac{2}{3}$ chord $1\text{–}2 \cdot e$. Now chord $1\text{–}2 = (e \cdot d)^{\frac{1}{2}}$, approximately, and (73) becomes

$$S \cdot l(e \cdot d)^{\frac{1}{2}} = W \cdot e \tag{74}$$

Solving equations (71) and (74), we have

$$W = \tfrac{2}{3} l \cdot d \cdot S(2S/E)^{\frac{1}{2}} \tag{75}$$

or

$$w = \tfrac{2}{3} d \cdot S(2S/E)^{\frac{1}{2}} \tag{76}$$

where w is the load per lineal inch of roller, if d is given in inches.

Now taking $S = 15{,}000$ lbs. per sq. in., and $E = 30{,}000{,}000$ lbs. per sq. in., equation (76) becomes

$$w = 315d \tag{77}$$

CAMBER.—Bridges are constructed so that when loaded the trusses will take the form assumed in the calculations. This may be done in two ways: (1) by increasing the lengths of all compression members and decreasing the lengths of all tension members the amounts calculated as in column 6 in Table XI—this method requires laborious calculations and increases the labor in making and checking the drawings; (2) the most common method is to increase the lengths of the top chords $\frac{1}{8}$ in. in 10 ft. for railway bridges and $\frac{3}{16}$ in. in 10 ft. for highway bridges over the lengths of the lower chords. This method is very easy to apply and satisfies theoretical requirements quite closely.

Let $c =$ camber in inches at the center of the span;

$a =$ total increase in the length of the top chord required to produce the camber, in inches;

h = the height of the truss in feet;

l = the length of span in feet;

then

$$c = a \cdot l/8h, \text{ and } a = 8c \cdot h/l \tag{78}$$

Now in the 160-ft. span in Fig. 111, $c = 1.05$ ins., $l = 160' \ 0''$, and $h = 21' \ 0''$. Then

$$a = 8 \times 1.05 \times 21/160 = 1.12 \text{ inches.}$$

Now this increase will be put in 4 panels, giving 0.28 ins. in each panel, or 0.14 ins. in 10 ft. This is slightly less than $\frac{3}{16}$ in. for each 10 ft. as commonly specified for highway bridges.

CHAPTER IX.

THE SOLUTIONS OF PROBLEMS IN THE CALCULATION OF STRESSES IN BRIDGE TRUSSES.

Introduction.—To obtain a thorough knowledge of the calculation of stresses in trusses it is necessary to solve numerous problems. The problems in this chapter have been selected with care and have shown their value by actual use in the class-room. A problem is first solved and the solution is followed through in detail. A second problem of a similar character is then stated and left for the student to solve. These problems should be solved in connection with the study of the preceding chapters.

Instructions.—(1) *Plate:* The standard plate is to be $9'' \times 10\frac{1}{2}''$, with a $1''$ border on the left-hand side and a $\frac{1}{2}''$ border on the top, bottom and right-hand side of the plate. The plate inside the border is to be $7\frac{1}{2}'' \times 9\frac{1}{2}''$. (2) *Coördinates:* Unless stated to the contrary, the coördinates given in the data will refer to the lower left-hand corner of the plate as the origin of coördinates. (3) *Data:* Complete data shall be placed on each problem so that the solution will be self-explanatory. The span, panel length, depth, roadway and other dimensions shall be shown on the truss diagram and shall be stated in a prominent place. The loads shall be stated, and the values of all trigonometric functions shall be given to three decimal places. (4) *Lettering:* All lettering shall be in Engineering News style. The main headings shall be made with capitals $0.2''$ high, and lower case letters $\frac{2}{3}$ of this height. Capitals in the body of the problem are to be $0.15''$ in height, and the lower case letters are to be $\frac{2}{3}$ of this height. (5) *Scales:* The scale of the forces and of the trusses shall be given as $1'' = (\text{——})$ lbs., or ft.; and by a graphic scale as well. (6) *Name:* The name of the student is to be placed outside the border in the lower right-hand corner. (7) *Equations:* All equations shall be given, but details of the solution may be indicated. (8) *References:* References are to "The Design of Highway Bridges."

NOTE.—It should be noted that all the problems have been reduced so that all dimensions are one-half the original dimensions given in the statements of the problems.

Problem 1. Dead Load Stresses in a Warren Truss by Graphic Resolution.

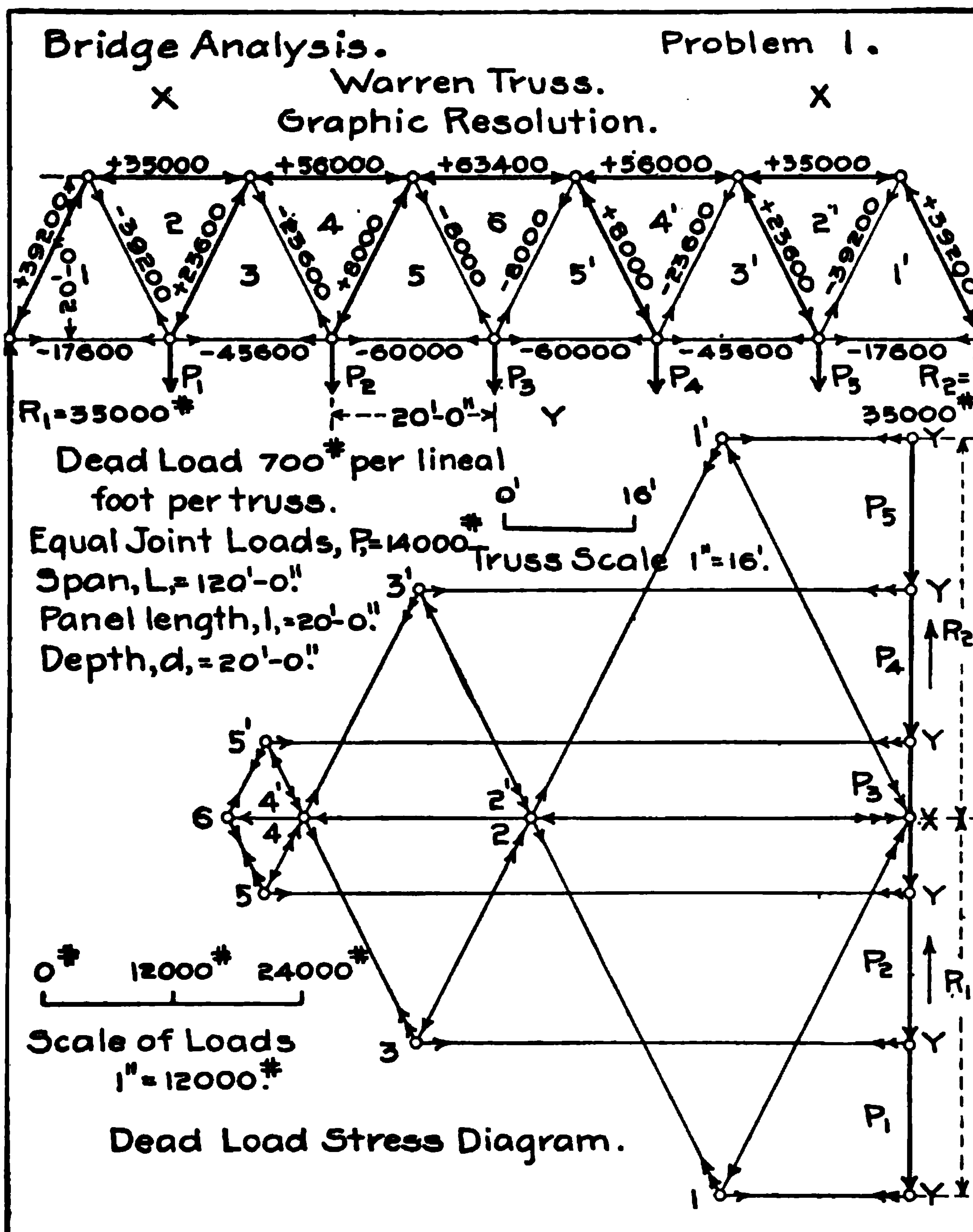
(*a*) **Problem.**—Given a Warren truss, span 120′ 0″, panel length 20′ 0″, depth 20′ 0″, dead load 700 lbs. per ft. per truss. Calculate the dead load stresses by graphic resolution. Scale of truss, 1″ = 16′ 0″. Scale of loads, 1″ = 12,000 lbs.

(*b*) **Methods.**—The loads beginning with the first load on the left are laid off from the bottom upwards. The calculation of the stresses is started at the left reaction, and the stress diagram is closed at the right reaction. For additional information on the solution see Chapter V.

(*c*) **Results.**—The top chord is in compression, the bottom chord is in tension; all web members leaning toward the center of the truss are in compression, while the web members leaning toward the abutments are in tension. All web members meeting on the unloaded chord (top chord) have stresses equal in amount but opposite in sign. The stresses in the lower chord are the arithmetical means of the stresses in adjacent panels of the top chord. Warren trusses are commonly made of iron or steel with riveted connections, the most common section being two angles placed back to back.

Problem 1a. Dead Load Stresses in a Warren Truss by Graphic Resolution.

(*a*) **Problem.**—Given a Warren truss, span 140′ 0″, panel length 20′ 0″, depth 24′ 0″, dead load 600 lbs. per ft. per truss. Calculate the dead load stresses by graphic resolution. Scale of truss, 1″ = 20′ 0″. Scale of loads, 1″ = 12,000 lbs.


Bridge Analysis.
Problem 1.
Warren Truss.
Graphic Resolution.
+35000
+56000
+63400
+56000
+35000
-17600
-45600
-60000
-60000
-45600
-17600
R1=35000#
R2=35000#
Dead Load 700# per lineal foot per truss.
Equal Joint Loads, P=14000#
Span, L, = 120'-0"
Panel length, l, = 20'-0"
Depth, d, = 20'-0"
Truss Scale 1"=16'.
0# 12000# 24000#
Scale of Loads 1" = 12000#
Dead Load Stress Diagram.

PROBLEM 2. DEAD LOAD STRESSES IN A PRATT TRUSS BY GRAPHIC RESOLUTION.

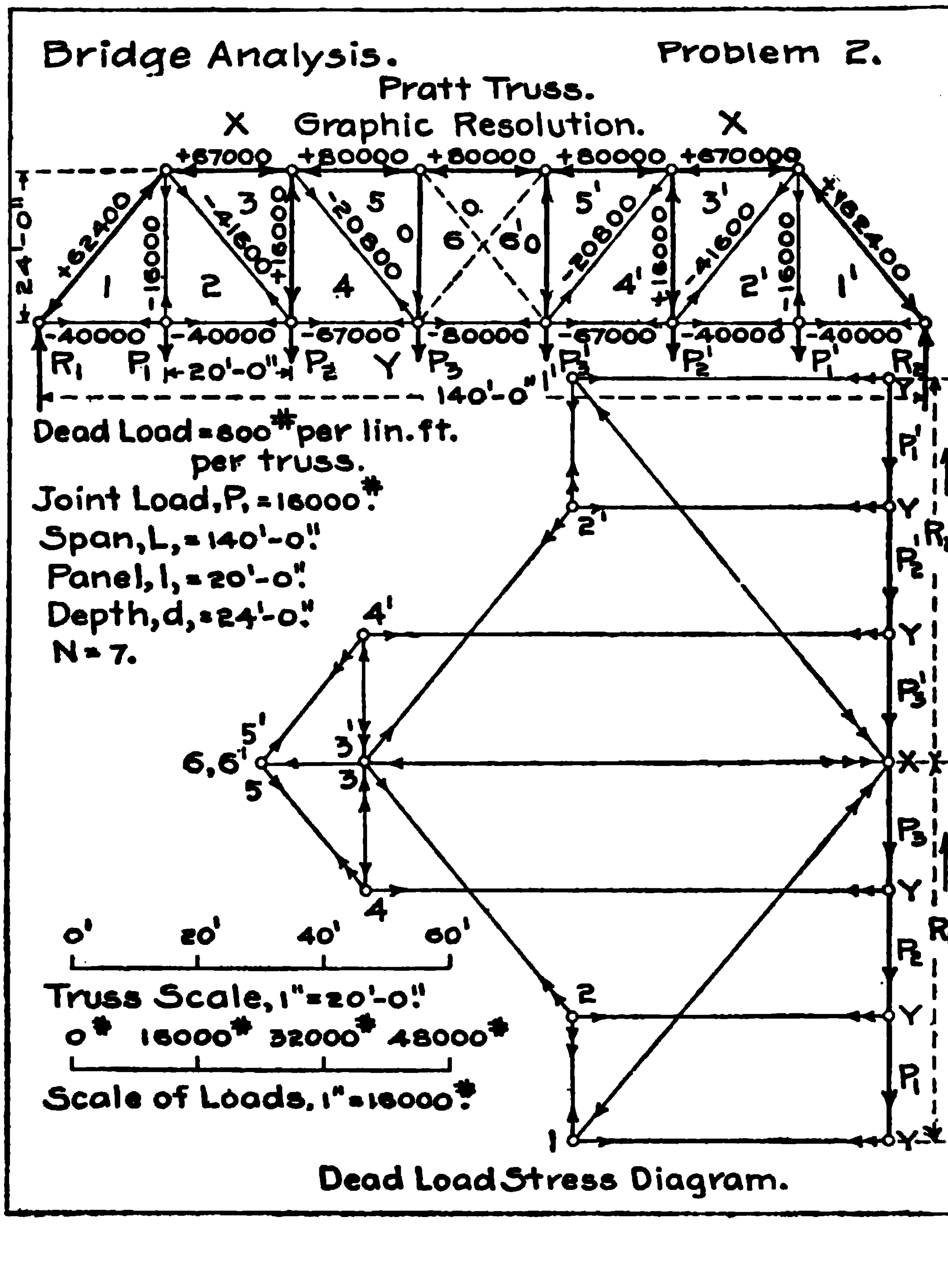
(*a*) **Problem.**—Given a Pratt truss, span 140′ 0″, panel length 20′ 0″, depth 24′ 0″, dead load 800 lbs. per lineal foot per truss. Calculate the dead load stresses by graphic resolution. Scale of truss, 1′ = 20′ 0″. Scale of loads, 1″ = 16,000 lbs.

(*b*) **Methods.**—The loads beginning with the first load on the left are laid off from the bottom upwards. Calculate the stresses by graphic resolution, beginning at R_1 and checking up at R_2, following the order shown in the stress diagram.

(*c*) **Results.**—The top chord is in compression and the bottom chord is in tension as in the Warren truss. The inclined members are in tension, while the vertical members are in compression. Member 1–2 is simply a hanger. There is no stress due to dead loads in the diagonal members in the middle panel of a Pratt truss with an odd number of panels. The stresses in the posts are equal to the inclined components of the stresses in the inclined members, meeting them on the unloaded chord (top chord). Stresses in certain panels in the top and bottom chord are equal. The Pratt truss is quite generally used for steel bridges and is also used for combination bridges, where the tension members are made of iron or steel and the compression members are made of timber.

PROBLEM 2A. DEAD LOAD STRESSES IN A PRATT TRUSS BY GRAPHIC RESOLUTION.

(*a*) **Problem.**—Given a Pratt truss, span 160′ 0″, panel length 20′ 0″, depth 24′ 0″, dead load 800 lbs. per lineal foot per truss. Calculate the dead load stresses by graphic resolution. Scale of truss, 1″ = 25′ 0″. Scale of loads, 1″ = 20,000 lbs.


Bridge Analysis.
Problem 2.
Pratt Truss.
Graphic Resolution.
X
X
+67000
+80000
+80000
+80000
+67000
+62400
-16000
-41600
+16000
-20800
6
6'
-20800
+16000
-41600
-16000
+62400
-40000
-40000
-67000
-80000
-67000
-40000
-40000
R_1
P_1
20'-0"
P_2
Y
P_3
P_3'
P_2'
P_1'
R_2
140'-0"
24'-0"
Dead Load = 800# per lin. ft. per truss.
Joint Load, P, = 16000#
Span, L, = 140'-0"
Panel, l, = 20'-0"
Depth, d, = 24'-0"
N = 7.
0'
20'
40'
60'
Truss Scale, 1" = 20'-0"
0#
16000#
32000#
48000#
Scale of Loads, 1" = 16000#
6,6'
Dead Load Stress Diagram.

PROBLEM 3. DEAD LOAD STRESSES IN A HOWE TRUSS BY GRAPHIC RESOLUTION.

(*a*) **Problem.**—Given a Howe truss, span 160′ 0″, panel length 20′ 0″, depth 24′ 0″, dead load 600 lbs. per lineal foot per truss. Calculate the dead load stresses by graphic resolution. Scale of truss, 1″ = 25′ 0″. Scale of loads, 1″ = 15,000 lbs.

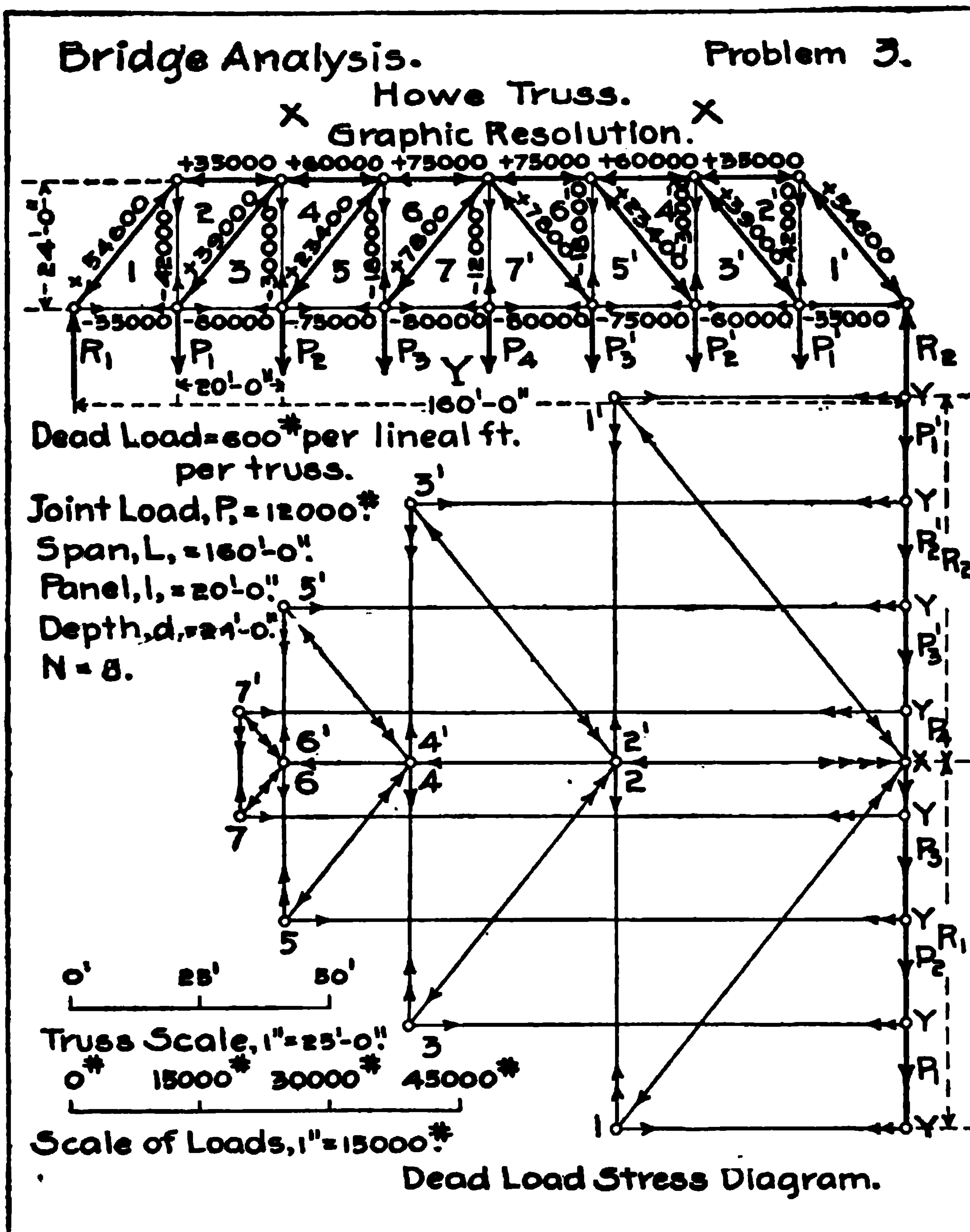
(*b*) **Methods.**—The loads beginning with the first load on the left are laid off from the bottom upwards. Calculate the stresses by graphic resolution, beginning at R_1 and checking at R_2, following the order shown in the stress diagram.

(*c*) **Results.**—The top chord is in compression and the bottom chord is in tension as in the Warren truss. All inclined members are in compression, while all vertical members are in tension. The stresses in the verticals are equal to the vertical components of the stresses in the diagonal members meeting them on the unloaded chord. Stresses in certain panels in the top and bottom chord are equal.

The Howe truss when used for highway or railroad bridges is commonly built with timber top and bottom chords and timber diagonal struts, the only iron being the vertical ties and the cast iron angle blocks to take the bearing of the timber struts. This makes a very satisfactory truss and is quite economical where timber is cheap.

PROBLEM 3A. DEAD LOAD STRESSES IN A HOWE TRUSS BY GRAPHIC RESOLUTION.

(*a*) **Problem.**—Given a Howe truss, span 162′ 0″, panel length 18′ 0″, depth 24′ 0″, dead load 600 lbs. per lineal foot per truss. Calculate the dead load stresses by graphic resolution. Scale of truss, 1″ = 25′ 0″. Scale of loads, 1″ = 15,000 lbs.


Bridge Analysis.
Problem 3.
Howe Truss.
Graphic Resolution.
+35000 +60000 +75000 +75000 +60000 +35000
-35000 -60000 -75000 -80000 -80000 -75000 -60000 -35000
R1 P1 P2 P3 Y P4 P3' P2' P1' R2
20'-0"
160'-0"
Dead Load=600# per lineal ft. per truss.
Joint Load, P,=12000#
Span, L, =160'-0"
Panel, l, =20'-0"
Depth, d, =24'-0"
N=8.
0' 25' 50'
Truss Scale, 1"=25'-0"
0# 15000# 30000# 45000#
Scale of Loads, 1"=15000#
Dead Load Stress Diagram.

PROBLEM 4. DEAD LOAD STRESSES IN A CAMEL-BACK TRUSS BY GRAPHIC RESOLUTION.

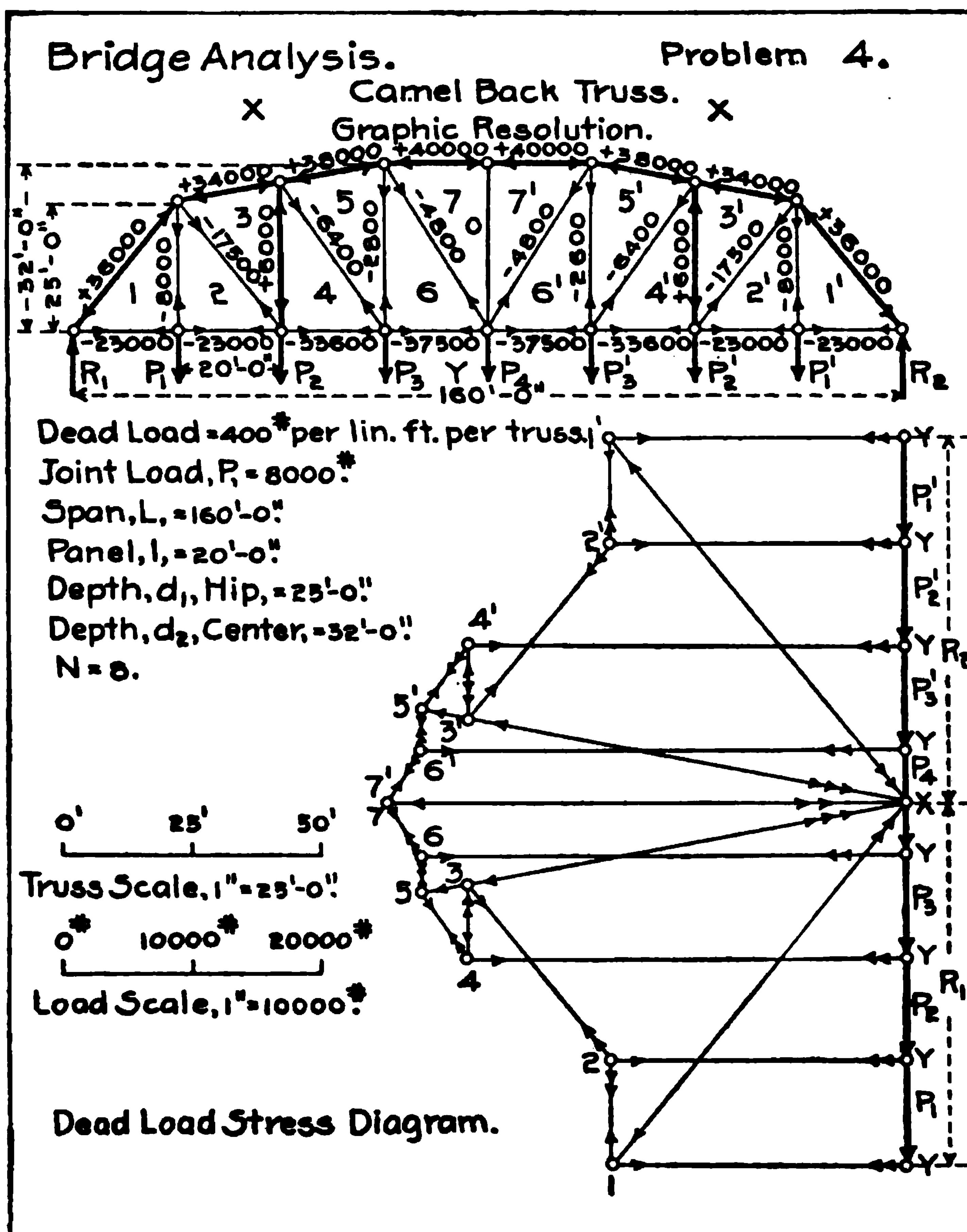
(*a*) **Problem.**—Given a Camel-back (inclined Pratt) truss, span 160′ 0″, panel length 20′ 0″, depth at the hip 25′ 0″, depth at the center 32′ 0″, dead load 400 lbs. per lineal foot per truss. Calculate the dead load stresses by graphic resolution. Scale of truss, 1″ = 25′ 0″. Scale of loads, 1″ = 10,000 lbs.

(*b*) **Methods.**—The loads beginning with the first load on the left are laid off from the bottom upwards. Calculate the stresses by graphic resolution, beginning at R_1 and checking up at R_2. Follow the order given in the stress diagram.

(*c*) **Results.**—The top chord is in compression and the bottom chord is in tension the same as in the Pratt truss. All inclined web members are in tension; while part of the posts are in compression and part are in tension. Member 1–2 is simply a hanger and is always in tension. This type of truss is quite generally used for steel and combination bridges for spans from 150 to 200 feet, and also for long span roof trusses. In the roof truss, the loads are on both the top and bottom chords or on the top chord alone.

PROBLEM 4A. DEAD LOAD STRESSES IN A CAMEL-BACK TRUSS BY GRAPHIC RESOLUTION.

(*a*) **Problem.**—Given a Camel-back (inclined Pratt) truss, span 180′ 0″, panel length 20′ 0″ (three panels with parallel chords), depth at the hip 25′ 0″, depth at the center 32′ 0″, dead load 400 lbs. per lineal foot per truss. Calculate the dead load stresses by graphic resolution. Scale of truss, 1″ = 25′ 0″. Scale of loads, 1″ = 12,000 lbs.


Bridge Analysis.
Problem 4.
Camel Back Truss.
Graphic Resolution.
Dead Load = 400# per lin. ft. per truss.
Joint Load, P_1 = 8000#.
Span, L, = 160'-0".
Panel, l, = 20'-0".
Depth, d_1, Hip, = 25'-0".
Depth, d_2, Center, = 32'-0".
N = 8.
Truss Scale, 1" = 25'-0".
Load Scale, 1" = 10000#.
Dead Load Stress Diagram.

PROBLEM 5. DEAD LOAD STRESSES IN A BALTIMORE TRUSS BY GRAPHIC RESOLUTION.

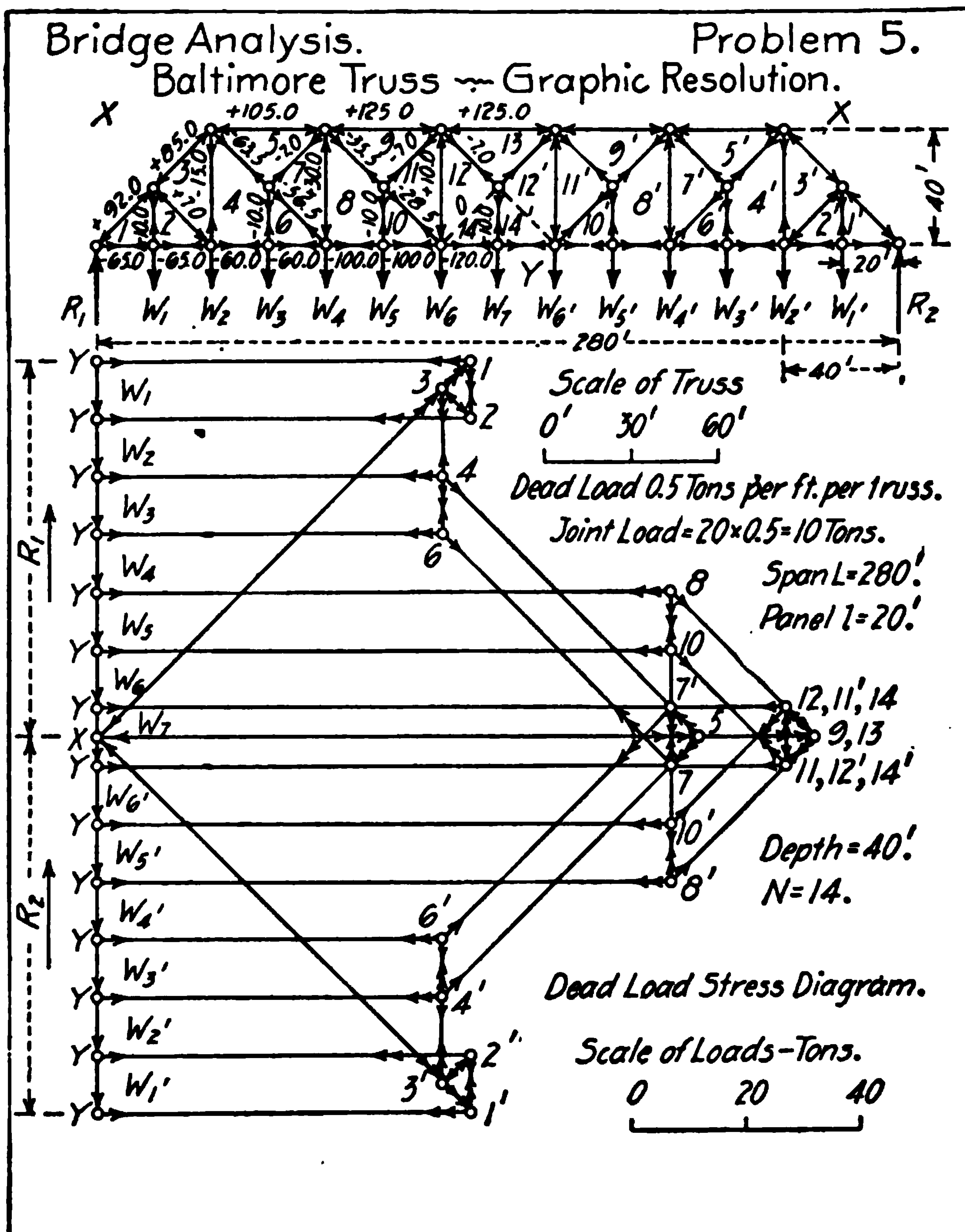
(*a*) **Problem.**—Given a Baltimore truss, span 280′ 0″, panel length 20′ 0″, depth 40′ 0″, dead load 0.5 tons per lineal foot per truss. Calculate the dead load stresses by graphic resolution. Scale of truss, 1″ = 40′ 0″. Scale of loads, 1″ = 40 tons.

(*b*) **Methods.**—The loads beginning with the first load on the left are laid off from the top downwards. Calculate R_1 and R_2. Calculate the stresses at the left reaction by constructing triangle 1–Y–X as shown. Then calculate the stress in 1–2 by constructing polygon Y–1–2–Y. Draw 3–2, which is the stress in member 3–2. Then calculate the stress in 3–4 and 4–Y by constructing polygon Y–2–3–4–Y. Calculate the stresses in the remaining members in order, finally checking up at R_2.

(*c*) **Results.**—It will be seen that the Baltimore truss is a Pratt truss with subdivided panels. The stresses in the first and second panels of the lower chord are larger than the stresses in the third and fourth panels of the lower chord. The stress in 6–7 is equal to the inclined component of the shear in the panel, plus the stress due to the half load that is carried toward the center of the bridge by 5–7. The Baltimore truss is used for long spans in which short panels can be used with an economical depth.

PROBLEM 5A. DEAD LOAD STRESSES IN A BALTIMORE TRUSS BY GRAPHIC RESOLUTION.

(*a*) **Problem.**—Given a Baltimore truss, span 320′ 0″, panel length 20′ 0″, depth 50′ 0″, dead load 0.5 tons per lineal foot per truss. Calculate the dead load stresses by graphic resolution. Scale of truss, 1″ = 50′ 0″. Scale of loads, 1″ = 50 tons.

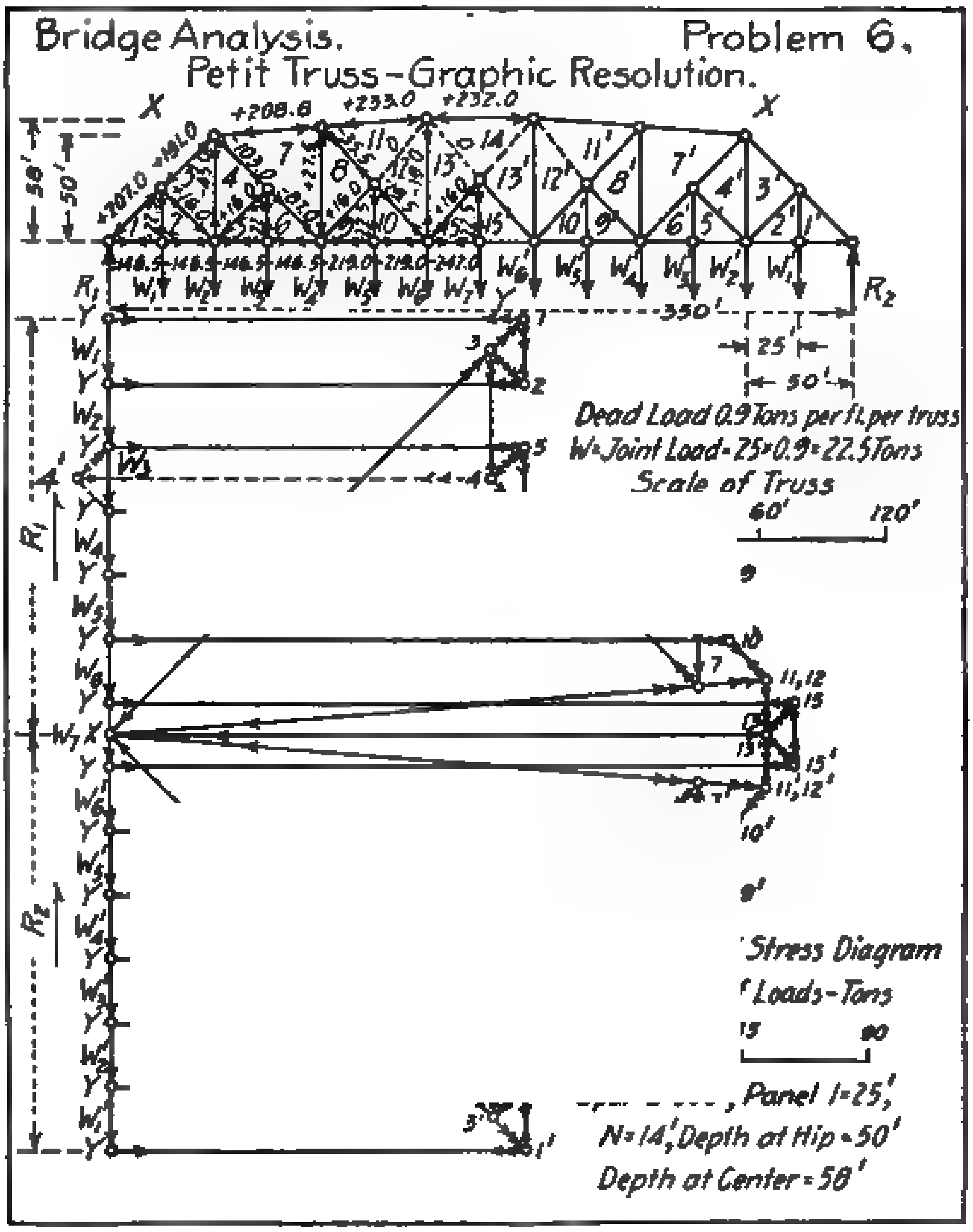

Bridge Analysis.
Problem 5.
Baltimore Truss — Graphic Resolution.
Scale of Truss
0' 30' 60'
Dead Load 0.5 Tons per ft. per truss.
Joint Load = 20×0.5 = 10 Tons.
Span L = 280'.
Panel l = 20'.
Depth = 40'.
N = 14.
Dead Load Stress Diagram.
Scale of Loads - Tons.
0 20 40

PROBLEM 6. DEAD LOAD STRESSES IN A PETIT TRUSS BY GRAPHIC RESOLUTION.

(*a*) **Problem.**—Given a Petit truss, span 350′ 0″, panel length 25′ 0″, depth at hip 50′ 0″, depth at center 58′ 0″, dead load 0.9 tons per lineal foot per truss. Calculate the dead load stresses by graphic resolution. Scale of truss, 1″ = 50′ 0″. Scale of loads, 1″ = 45 tons.

(*b*) **Methods.**—The loads beginning with the first load on the left are laid off from the top downwards. Calculate R_1 and R_2. Calculate the stresses in the members at the left reaction by constructing force triangle 1–Y–X. Then calculate the stress in 1–2 by constructing polygon Y–1–2–Y. Draw 3–2, which is the stress in member 3–2. Then pass to joint W_2 where there appears to be an ambiguity, stress 4–5 being unknown. To remove the ambiguity proceed as follows: At W_3 on the left side of the stress diagram assume that W_3 is the stress in 5–6 (the member 5–6 is simply a hanger and the stress is as assumed). Calculate the stress in 4–5 by completing the triangle of stresses in the auxiliary members. The stresses are now all known at W_2 except 3–4 and 5–Y, but the stress in 4–5 is between the two unknown stresses. First complete the force polygon 2–3–4–5′–Y–Y–2. Then by changing the order the true polygon 2–3–4–5–Y–Y–2 may be drawn. This solution is sometimes called the method of sliding in a member. The apparent ambiguity at joint W_4 may be removed in the same manner. The stress diagram is carried through as shown and finally checked up at R_2. It will be seen that there is no apparent ambiguity on the right side of the truss.

(*c*) **Results.**—It will be seen that the Petit truss is an inclined Pratt or Camel-back truss with subdivided panels. The auxiliary members are commonly tension members in all except the end primary panels as in the Baltimore truss in Problem 5. It will be seen that the stresses in the first four panels of the lower chord are the same. The loads in this type of Petit truss are carried directly to the abutments. The Petit truss is quite generally used for long span highway and railway bridges.



Problem 6a. Dead Load Stresses in a Petit Truss by Graphic Resolution.

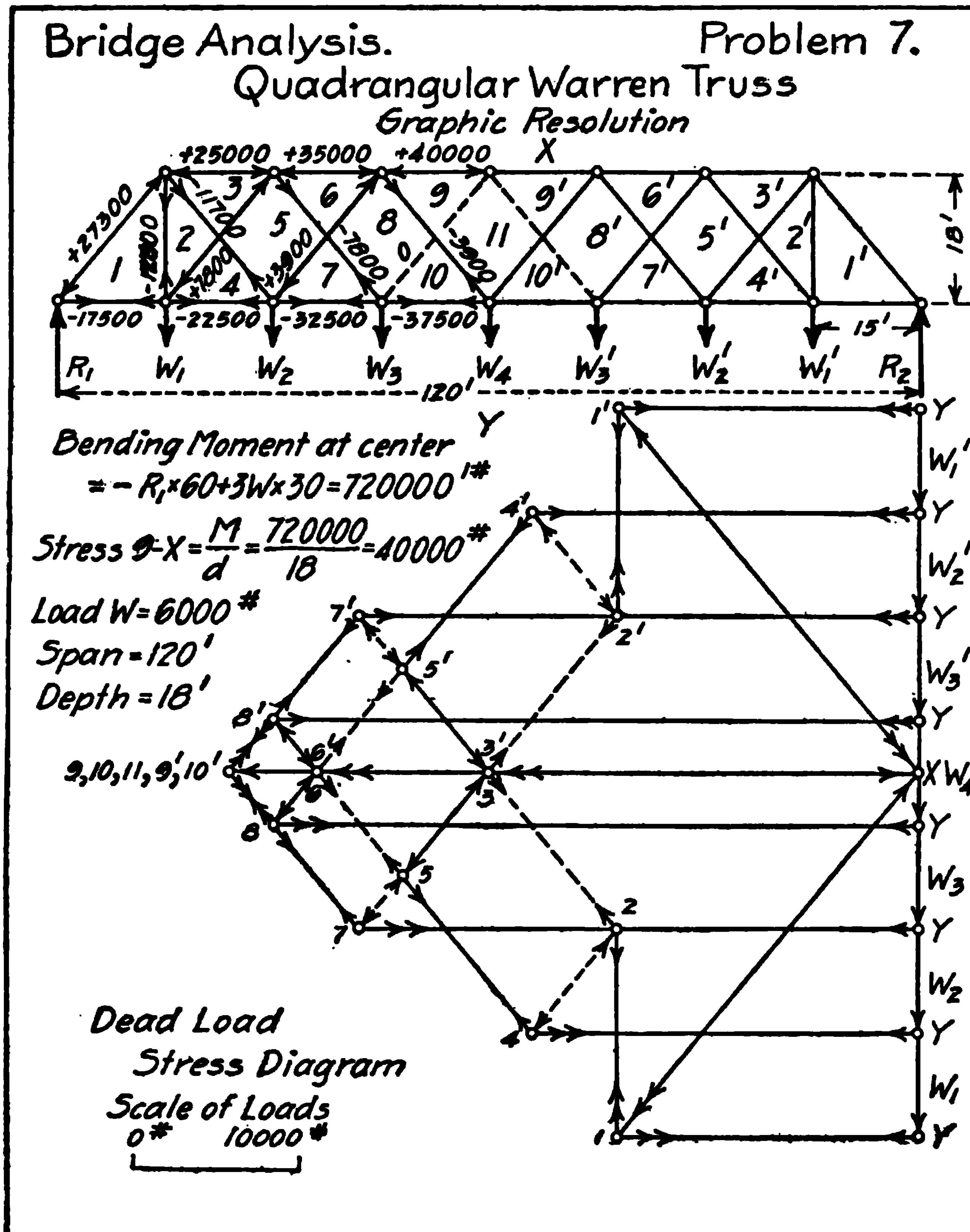
(*a*) **Problem.**—Given a Petit truss with the same span, panel length, depths, and dead load as in Problem 6; the auxiliary members being arranged as in the Baltimore truss in Problem 5.

PROBLEM 7. DEAD LOAD STRESSES IN A QUADRANGULAR WARREN TRUSS BY GRAPHIC RESOLUTION.

(*a*) **Problem.**—Given a quadrangular Warren truss, span 120′ 0″, panel length 15′ 0″, depth 18′ 0″, dead load 400 lbs. per lineal foot per truss. Calculate the dead load stresses by graphic resolution. Scale of truss and loads as shown.

(*b*) **Methods.**—The loads beginning with the first load on the left are laid off from the bottom upwards. Calculate R_1 and R_2. The stresses at R_1 may be calculated by constructing force polygon 1–X–Y. However, on passing to the next joint in either the top or bottom chord the solution is indeterminate. To solve the problem calculate the stress in the top chord 9–X by taking moments about the center joint in the bottom chord, the stress in 9–11 being zero. Lay off 9–X in the stress diagram and complete the diagram to the left and the right of the center as shown. It will be seen that the stresses in certain members occur twice in the diagram. The truss diagram can be divided into two systems as in Problem 14, and the stresses can be calculated for each system, the chord stresses in the two systems being added together for the final stresses. However, the stresses as calculated in this problem are the correct ones.

(*c*) **Results.**—The quadrangular Warren truss is a double intersection truss in which the stresses are statically determinate for dead loads but are statically indeterminate for live load web stresses, as will be shown in Problem 14. This truss, built with riveted connections, is extensively used by the American Bridge Company for highway bridges for spans from 80 to 152 feet.



PROBLEM 7A. DEAD LOAD STRESSES IN A QUADRANGULAR WARREN TRUSS BY GRAPHIC RESOLUTION.

(*a*) **Problem.**—Given a quadrangular Warren truss, span 150′ 0″, panel length 15′ 0″, depth 18′ 0″, dead load 500 lbs. per lineal foot per truss. Calculate the dead and live load stresses by graphic resolution. Scale of truss, 1″ = 20′ 0″. Scale of loads, 1″ = 10,000 lbs.

PROBLEM 8. DEAD LOAD STRESSES IN A WARREN TRUSS BY ALGEBRAIC RESOLUTION (METHOD OF COEFFICIENTS).

(*a*) **Problem.**—Given a Warren truss, span 140′ 0″, panel length 20′ 0″, depth 20′ 0″, dead load 800 lbs. per lineal foot per truss. Calculate the dead load stresses by algebraic resolution. Scale of truss, 1″ = 20′ 0″.

(*b*) **Methods.**—Beginning at the left the left reaction $R_1 = 3W$. The shear in the first panel is $3W$, in the second panel is $2W$, in the third panel is W, and in the fourth panel is zero. Now resolving at R_1 the stress in $1\text{-}Y = -3W \cdot \tan\theta$, stress $1\text{-}X = +3W \cdot \sec\theta$. Cut members $1\text{-}Y$, $1\text{-}2$ and $2\text{-}X$ and the truss to the right by a plane and equate the horizontal components of the stresses in the members. The unknown stress $2\text{-}X$ will equal the sum of the horizontal components of the stresses in $1\text{-}Y$ and $1\text{-}2$ with sign changed, $= -(-3-3)W \cdot \tan\theta = +6W \cdot \tan\theta$. The stress in $3\text{-}Y = -(6+2)W \cdot \tan\theta = -8W \cdot \tan\theta$. Stress in $4\text{-}X = -(-8-2)W \cdot \tan\theta = +10W \cdot \tan\theta$; stress in $5\text{-}Y = -(+10+1)W \cdot \tan\theta = +11W \cdot \tan\theta$; and the stress in $6\text{-}X = -(-11-1)W \cdot \tan\theta = +12W \cdot \tan\theta$. The coefficients of the chord stresses when multiplied by $W \cdot \tan\theta$ give the stresses, while the coefficients for the webs when multiplied by $W \cdot \sec\theta$ give the web stresses.

(*c*) **Results.**—In the method of coefficients the shears are calculated first, and the chord coefficients follow easily by summing the horizontal components. This method is the shortest and the best for the calculation of the stresses in bridge trusses with parallel chords.

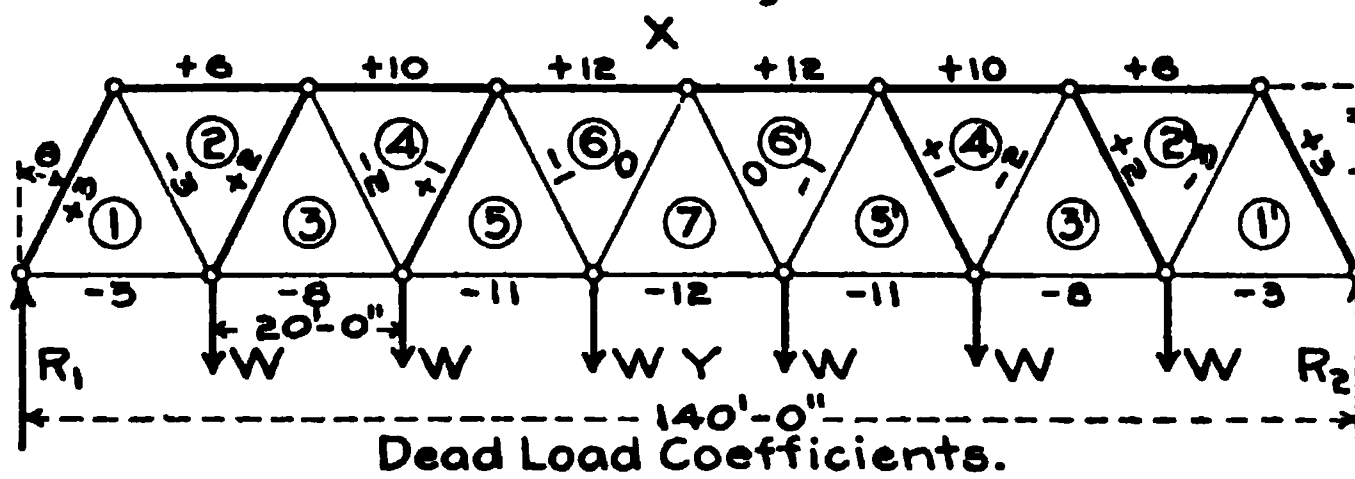
PROBLEM 8A. DEAD LOAD STRESSES IN A WARREN TRUSS BY ALGEBRAIC RESOLUTION.

(*a*) **Problem.**—Given a Warren truss, span 160′ 0″, panel length 20′ 0″, depth 24′ 0″, dead load 700 lbs. per lineal foot per truss. Calculate the dead load stresses by algebraic resolution. Scale of truss, 1″ = 25′ 0″.

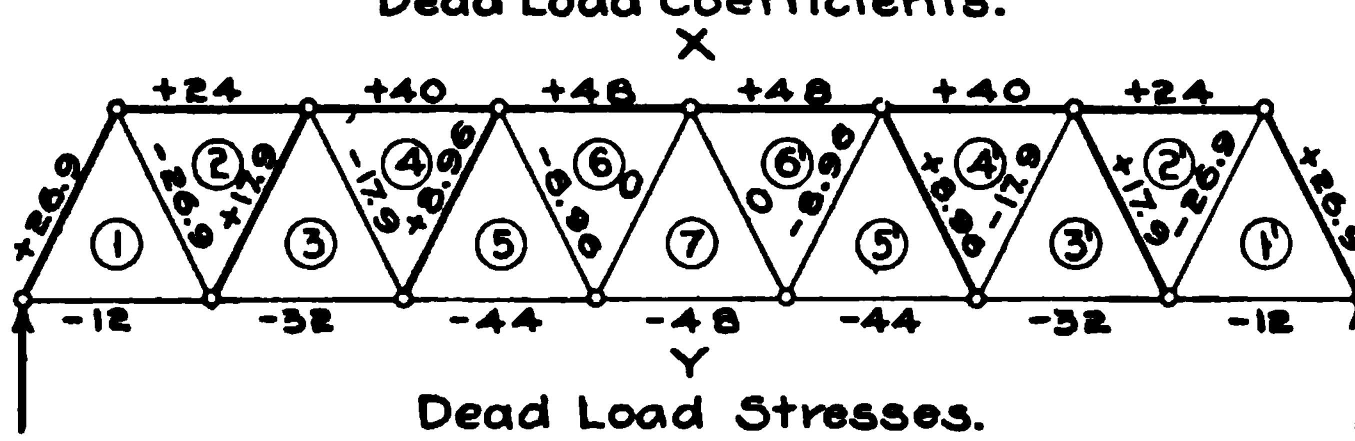
Bridge Analysis. Problem 8.

Warren Truss.

Dead Load Stresses. Algebraic Resolution.



Dead Load Coefficients.



Dead Load Stresses.

Joint Load, W, = 8 tons.

Chord Stresses = Coefficient × W × tan θ.

Web " = Coefficient × W × sec θ.

Span, L, = 140'-0". Sec θ = 1.12.

Panel length, l, = 20'-0". Tan θ = 0.50.

Depth, d, = 20'-0".

Stresses in Chords = Coeff. × W tan θ

" " Webs = " × W sec θ

PROBLEM 9. LIVE LOAD STRESSES IN A WARREN TRUSS BY ALGEBRAIC RESOLUTION.

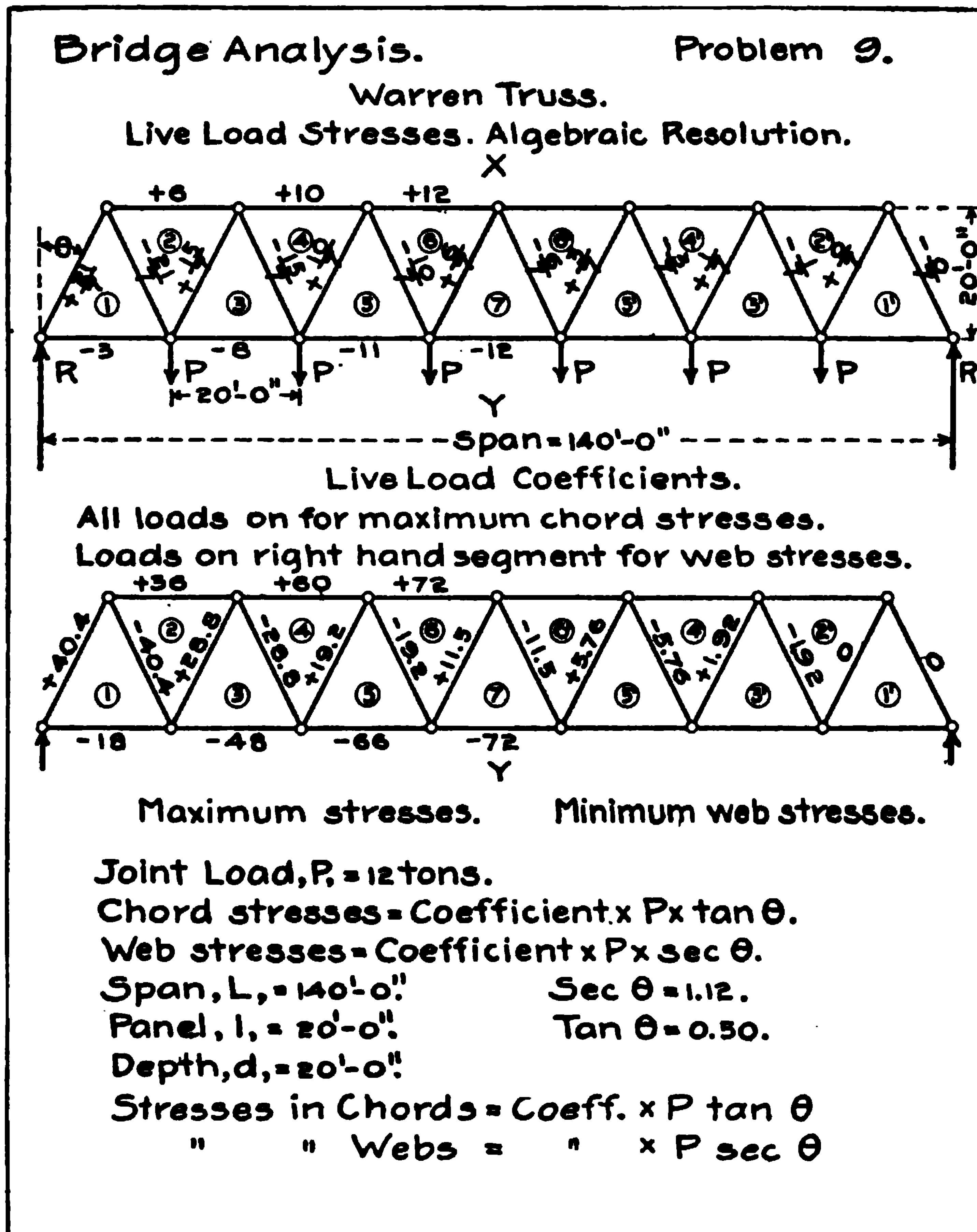
(*a*) **Problem.**—Given a Warren truss, span 140′ 0″, panel length 20′ 0″, depth 20′ 0″, live load 1,200 lbs. per lineal foot per truss. Calculate the maximum and minimum stresses due to the live load by algebraic resolution. Scale of truss, 1″ = 20′ 0″.

(*b*) **Methods.**—Construct two truss diagrams as shown. On the first place the live load coefficients, and on the second place the maximum and minimum live load stresses.

Chord Stresses.—The maximum chord stresses occur when the joints are all loaded, and the chord coefficients are found as in Problem 8. The minimum live load stresses in the chords occur when none of the joints are loaded, and are zero for each member.

Web Stresses.—The maximum web stresses in any panel occur when the longer segment into which the panel divides the truss is loaded, while the shorter segment has no loads on it. The minimum live load web stresses occur when the shorter segment is loaded and the longer segment has no loads on it. The maximum stresses in members 1–X and 1–2 occur when the truss is fully loaded. The shear in the panel is $3P$, or $21/7P$, and the stress in 1–$X = 3P \cdot \sec\theta = +40.4$ tons, while the stress in 1–2 $= -3P \cdot \sec\theta = -40.4$ tons. The minimum stresses in 1–X and 1–2 are zero. The maximum stresses in 2–3 and 3–4 occur when 5 loads are on the right of the panel and there are no loads on the left of the panel. The shear in the panel will then be equal to the left reaction, $= R_1 = (5 \times 3 \times P)/7 = 15/7P$. The stress in 2–3 $= 15/7P \cdot \sec\theta = +28.8$ tons, while the stress in 3–4 $= -15/7P \cdot \sec\theta = -28.8$ tons. The minimum stresses in 2–3 and 3–4 will occur when there is one load on the shorter segment. In the corresponding panel on the right of the truss, if the shorter segment is loaded, the left reaction $= 1/7P =$ the shear in the panel. The minimum stress in 2–3 $= -1/7P \cdot \sec\theta = -1.92$ tons, while the minimum stress in 3–4 $= +1.92$ tons. The stresses in the remaining panels are calculated in the same manner.

(*c*) **Results.**—It will be seen that the web members meeting on the unloaded chord (top chord) have their maximum and minimum stresses for the same loading. It will be seen that the maximum live load web stresses are of the same kind as the dead load web stresses, while the minimum web stresses are of the opposite kind. It will be seen that the numerators of the web coefficients from the right to the left are 0, 1, 3, 6, 10, 15, 21.



PROBLEM 9A. LIVE LOAD STRESSES IN A WARREN TRUSS BY ALGEBRAIC RESOLUTION.

(*a*) **Problem.**—Given a Warren truss, span 180′ 0″, panel length 20′ 0″, depth 24′ 0″, live load 1,500 lbs. per lineal foot per truss. Calculate the maximum and minimum stresses due to the live loads by algebraic resolution. Scale of truss, 1″ = 25′ 0″.

PROBLEM 10. MAXIMUM AND MINIMUM STRESSES IN A WARREN TRUSS BY ALGEBRAIC RESOLUTION.

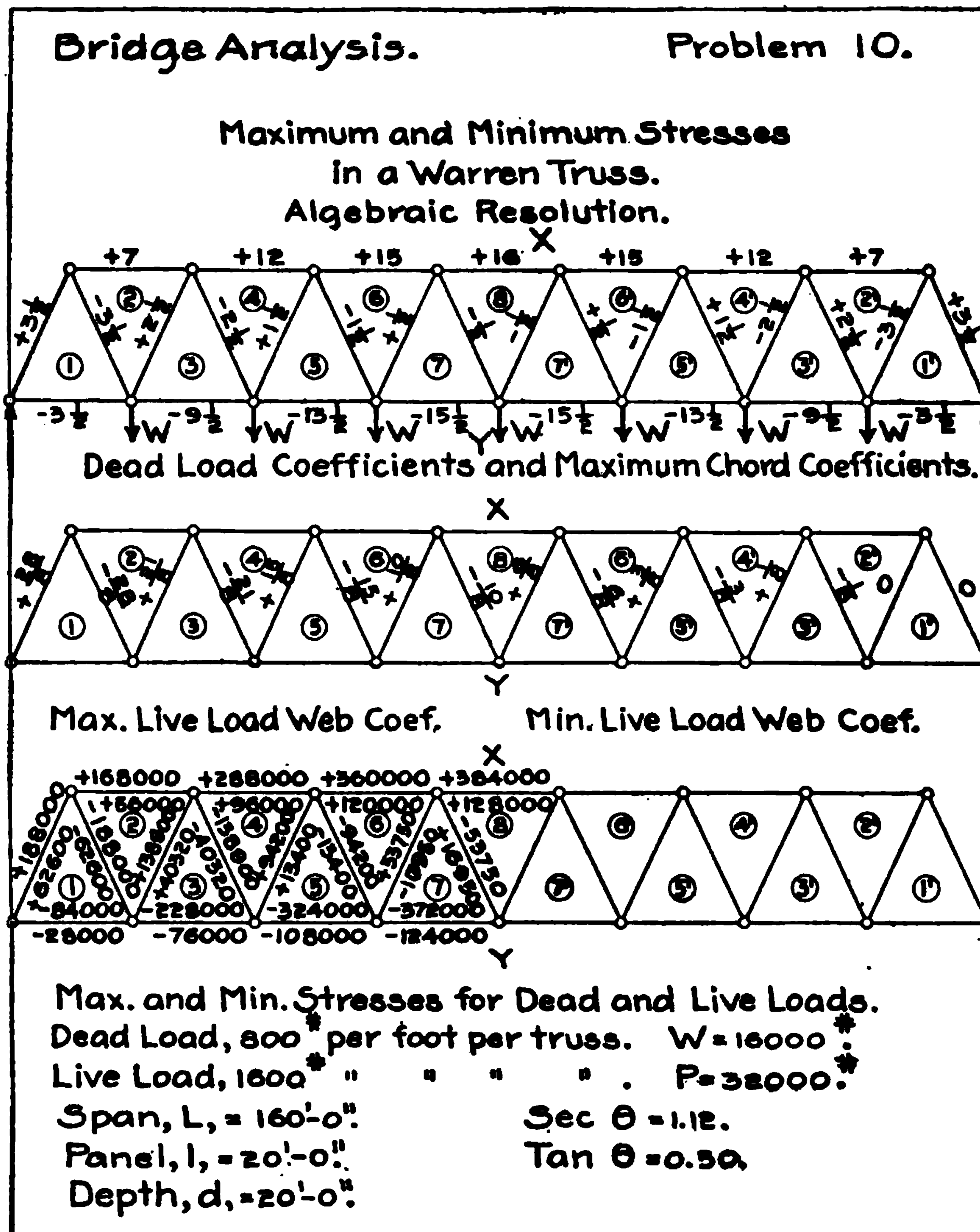
(*a*) **Problem.**—Given a Warren truss, span 160′ 0″, panel length 20′ 0″, depth 20′ 0″, dead load 800 lbs. per lineal foot per truss, live load 1,600 lbs. per lineal foot per truss. Calculate the maximum and minimum stresses in the members due to dead and live loads by algebraic resolution. Scale of truss as shown.

(*b*) **Methods.**—Construct three truss diagrams as shown. On the first truss diagram place the dead load and the maximum live load chord coefficients, calculated as in Problems 8 and 9. The maximum live load chord coefficients are the same as the dead load chord coefficients. On the second diagram place the maximum and minimum live load web coefficients, calculated as in Problem 9. The maximum live load web coefficients are given on the left and the minimum live load coefficients are given on the right of the diagram. On the third diagram place the maximum and minimum stresses. The maximum chord stresses are equal to the sum of the dead and live load chord stresses. The minimum chord stresses are the dead load chord stresses. The maximum web stresses are equal to the sum of the dead and the maximum live load web stresses. The minimum web stresses are equal to the algebraic sum of the dead load stresses and the minimum live load stresses.

(*c*) **Results.**—The web members 7–6 and 7–8 have a reversal of stress from tension to compression, or the reverse. These members must be counterbraced to take both kinds of stress.

PROBLEM 10A. MAXIMUM AND MINIMUM STRESSES IN A WARREN TRUSS BY ALGEBRAIC RESOLUTION.

(*a*) **Problem.**—Given a Warren truss, span 180′ 0″, panel length 20′ 0″, depth 24′ 0″, dead load 700 lbs. per lineal foot per truss, live load 1,500 lbs. per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads by algebraic resolution. Scale of truss, 1″ = 25′ 0″.

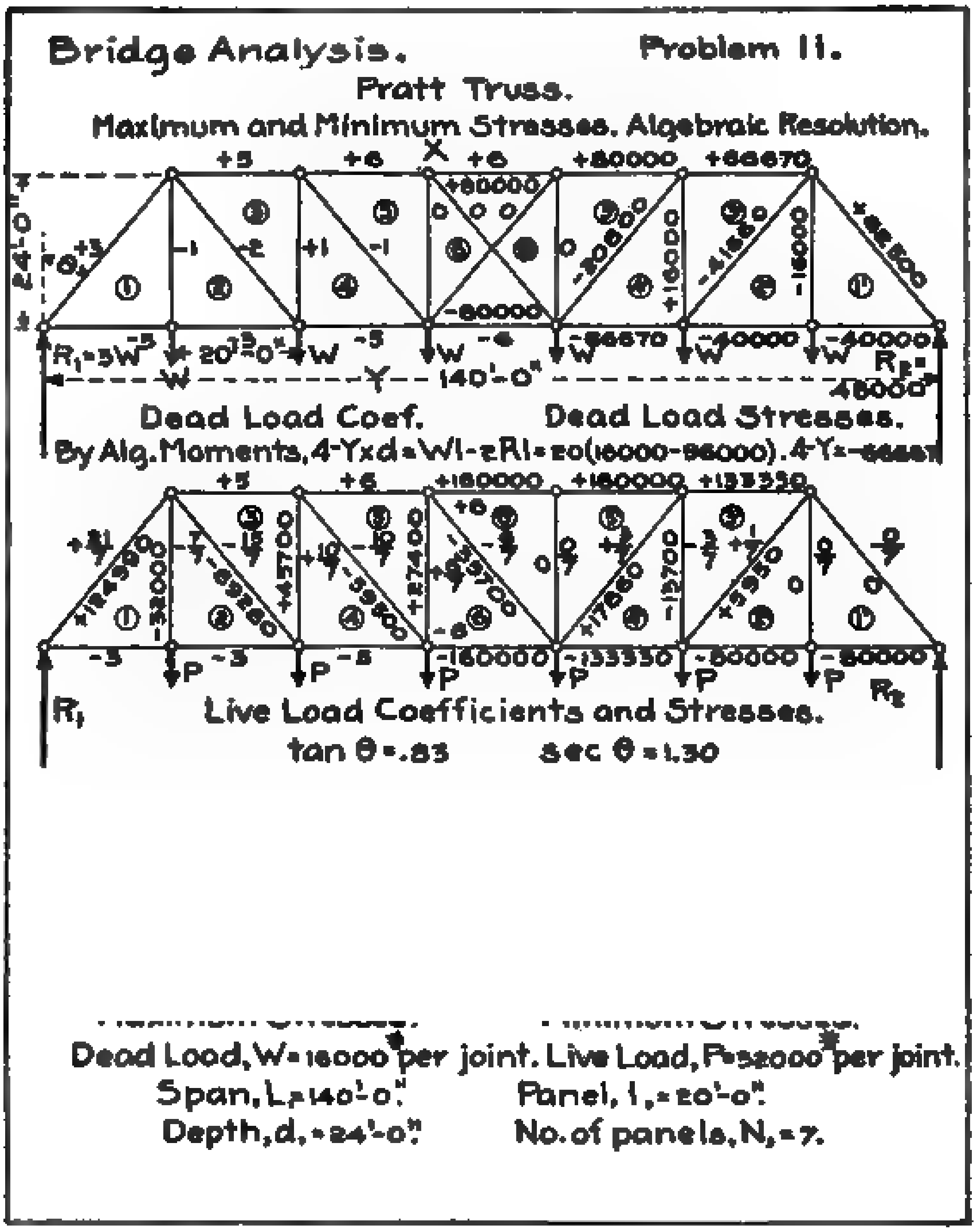

Bridge Analysis.
Problem 10.
Maximum and Minimum Stresses
in a Warren Truss.
Algebraic Resolution.
Dead Load Coefficients and Maximum Chord Coefficients.
Max. Live Load Web Coef.
Min. Live Load Web Coef.
+168000 +288000 +360000 +384000
-28000 -76000 -108000 -124000
Max. and Min. Stresses for Dead and Live Loads.
Dead Load, 800# per foot per truss. W = 16000#.
Live Load, 1600# " " " ". P = 32000#.
Span, L, = 160'-0".
Panel, l, = 20'-0".
Depth, d, = 20'-0".
Sec θ = 1.12.
Tan θ = 0.50.

PROBLEM 11. MAXIMUM AND MINIMUM STRESSES IN A PRATT TRUSS BY ALGEBRAIC RESOLUTION.

(*a*) **Problem.**—Given a Pratt truss, span 140′ 0″, panel length 20′ 0″, depth 24′ 0″, dead load 800 lbs. per lineal foot per truss, live load 1,600 lbs. per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads by algebraic resolution. Scale of truss, 1″ = 20′ 0″.

(*b*) **Methods.**—Construct three truss diagrams as shown. On the first place the dead load coefficients and the dead load stresses. On the second place the live load coefficients and the live load stresses. On the third place the maximum and minimum stresses due to dead and live loads. The maximum chord stresses are the sums of the dead and live load chord stresses, while the minimum chord stresses are those due to dead load alone. The hip vertical is simply a hanger and has a minimum stress of one dead load and a maximum stress of one live and one dead load. The conditions for maximum and minimum stresses in the webs are the same as for the Warren truss, the vertical posts having stresses equal to the vertical components of the stresses in the inclined web members meeting them on the unloaded (top) chord.

(*c*) **Results.**—There is no dead load shear in the middle panel, but it is seen that there are stresses in the counters for live loads. Only one of the counters will be in action at one time. Whenever the center of gravity of the loads is not in the center line of the truss, that counter will be acting that extends downward toward the center of gravity. The numerators of the maximum and minimum live load web coefficients are 0, 1, 3, 6, 10, 15, 21, as for the Warren truss. This shows that the maximum and minimum web stresses are proportional to the ordinates to a parabola.



PROBLEM 11A. MAXIMUM AND MINIMUM STRESSES IN A PRATT TRUSS BY ALGEBRAIC RESOLUTION.

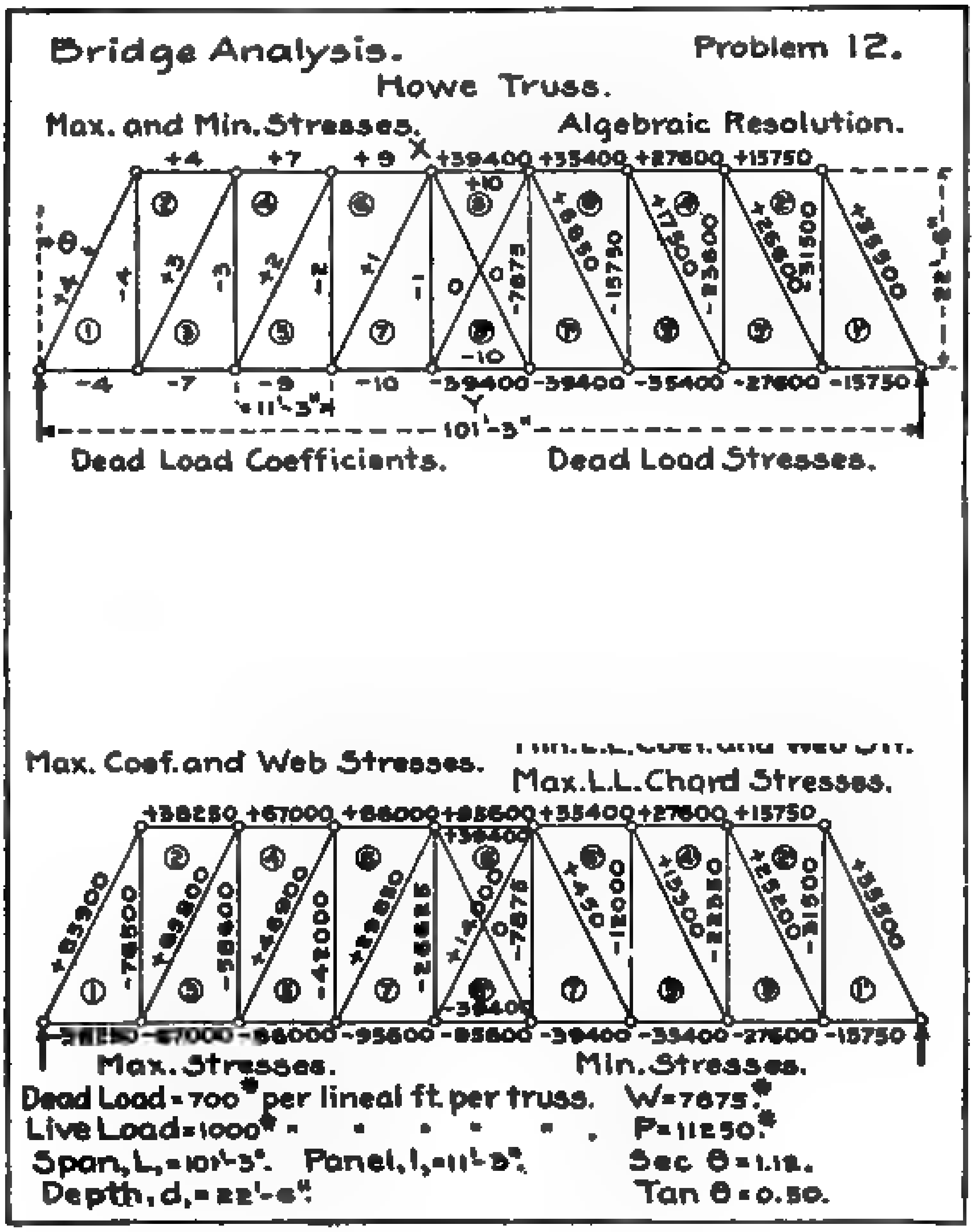
(*a*) **Problem.**—Given a Pratt truss, span 160′ 0″, panel length 20′ 0″, depth 26′ 0″, dead load 700 lbs. per lineal foot per truss, live load 1,500 lbs. per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads by algebraic resolution. Scale of truss, 1″ = 25′ 0″.

PROBLEM 12. MAXIMUM AND MINIMUM STRESSES IN A HOWE TRUSS BY ALGEBRAIC RESOLUTION.

(*a*) **Problem.**—Given a Howe truss, span 101′ 3″, panel length 11′ 3″, depth 22′ 6″, dead load 700 lbs. per lineal foot per truss, live load 1,000 lbs. per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads by algebraic resolution. Scale of truss, 1″ = 15′ 0″.

(*b*) **Methods.**—Construct three truss diagrams as shown. On the first diagram place the dead load coefficients and the dead load stresses. On the second diagram place the live load web coefficients and the maximum and minimum live load stresses. On the third diagram place the maximum and minimum stresses due to dead and live loads. The conditions for loading for the maximum and minimum stresses are the same as for a Pratt truss except that the vertical tie 1–2 carries the shear in the first panel and has a maximum stress for a full load on the truss.

(*c*) **Results.**—The vertical members are always in tension, while the diagonal members are always in compression. The web members meeting on the unloaded chord (top chord) have maximum and minimum stresses for the same loading. The counters in the center panel carry live load stress only, the counter acting downward away from the center of gravity of the loads being stressed. The maximum and minimum web stresses are the algebraic sums of the corresponding dead and live load stresses. The maximum chord stresses are the sums of the dead and live load chord stresses, while the minimum chord stresses are the dead load stresses alone.



PROBLEM 12A. MAXIMUM AND MINIMUM STRESSES IN A HOWE TRUSS BY ALGEBRAIC RESOLUTION.

(*a*) **Problem.**—Given a Howe truss, span 120′ 0″, panel length 12′ 0″, depth 24′ 0″, dead load 700 lbs. per lineal foot per truss, live load 1,000 lbs. per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads by algebraic resolution. Scale of truss, 1″ = 20′ 0″.

PROBLEM 13. MAXIMUM AND MINIMUM STRESSES IN A DECK BALTIMORE TRUSS BY ALGEBRAIC RESOLUTION.

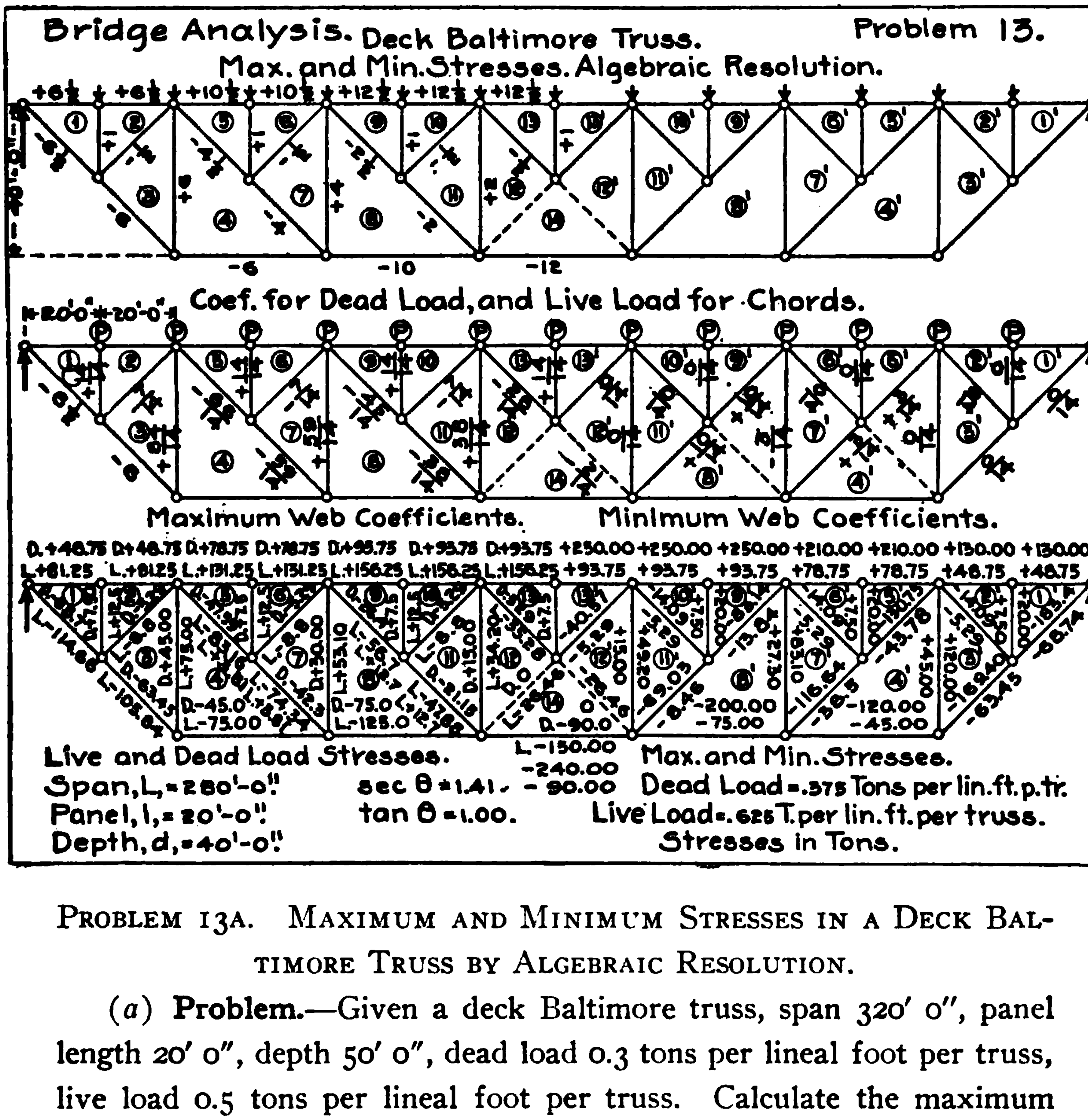
(*a*) **Problem.**—Given a deck Baltimore truss, span 280′ 0″, panel length 20′ 0″, depth 40′ 0″, dead load 0.375 tons per lineal foot per truss, live load 0.625 tons per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads by algebraic resolution.

(*b*) **Methods.**—Construct three truss diagrams and use them as shown.

Dead Load Stresses.—The auxiliary struts 1–2, 5–6, 9–10, etc., carry a full dead load compression, while the auxiliary web members 2–3, 6–7, 10–11, etc., have a tensile stress of $\frac{1}{2}W\cdot\sec\theta$. The stress in $1\text{–}Y$ equals the shear in the panel multiplied by $\sec\theta = -6\frac{1}{2}W\cdot\sec\theta$. The stress in $3\text{–}Y$ equals the shear in the panel multiplied by $\sec\theta$, plus the inclined component of the one-half load that is carried toward the center by the auxiliary member 2–3, $= -(5\frac{1}{2}+\frac{1}{2})W\cdot\sec\theta = -6W\cdot\sec\theta$. The stress in 3–4 is the vertical component of the stress in $3\text{–}Y = +6W$. The stress in $4\text{–}Y$ is the horizontal component of the stress in $3\text{–}Y = -6W\cdot\tan\theta$. The stress in $1\text{–}X$ and $2\text{–}X = +6\frac{1}{2}W\cdot\tan\theta$. The stress in 4–5 is the inclined component of the shear in the panel $= -4\frac{1}{2}W\cdot\sec\theta$. The stress in $5\text{–}X = -(-6-4\frac{1}{2})W\cdot\tan\theta = +10\frac{1}{2}W\cdot\tan\theta$. The remaining dead load stresses are calculated in a similar manner.

Live Load Web Stresses.—The maximum shears in the different panels occur when the longer segment of the truss is loaded, while the minimum shears occur when the shorter segment of the truss is loaded. The maximum stresses in the webs in the first and second panels occur for a full live load on the bridge. The maximum shear in the third panel occurs with all loads to the right of the panel and no loads to the left. The shear in the panel will then be equal to the left reaction $= 11 \times \frac{1}{2}(11+1)P/14 = 66/14P$. The maximum live load stress in 4–5 will be $= -66/14P\cdot\sec\theta$. With a maximum stress in 4–5 the stress in 4–7 will be $=(-66/14+7/14)P\cdot\sec\theta = -59/14P\cdot\sec\theta$. This is the maximum stress, for the stress in 4–7 when there is a maximum shear in the panel is $= 10\times 11/2\times 1/14P\cdot\sec\theta = -55/14P\cdot\sec\theta$. In a similar manner it will be found that maximum stresses in members 8–9 and 8–11 occur with a maximum shear in 8–9. On the right side it will be seen that minimum stresses in the diagonals occur for a minimum shear in the odd-numbered panels from the right.

(*c*) **Results.**—The dead and live loads were assumed as applied on the upper chord. The upper chords are in compression, while the lower chords are in tension the same as for a through truss. The live and dead load stresses are given separately on the left side of the lower truss as required by Cooper's Specfications.



PROBLEM 13A. MAXIMUM AND MINIMUM STRESSES IN A DECK BALTIMORE TRUSS BY ALGEBRAIC RESOLUTION.

(*a*) **Problem.**—Given a deck Baltimore truss, span 320′ 0″, panel length 20′ 0″, depth 50′ 0″, dead load 0.3 tons per lineal foot per truss, live load 0.5 tons per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads by algebraic resolution. Scale of truss, 1″ = 40′ 0″.

PROBLEM 14. MAXIMUM AND MINIMUM STRESSES IN A QUADRANGULAR WARREN TRUSS BY ALGEBRAIC RESOLUTION.

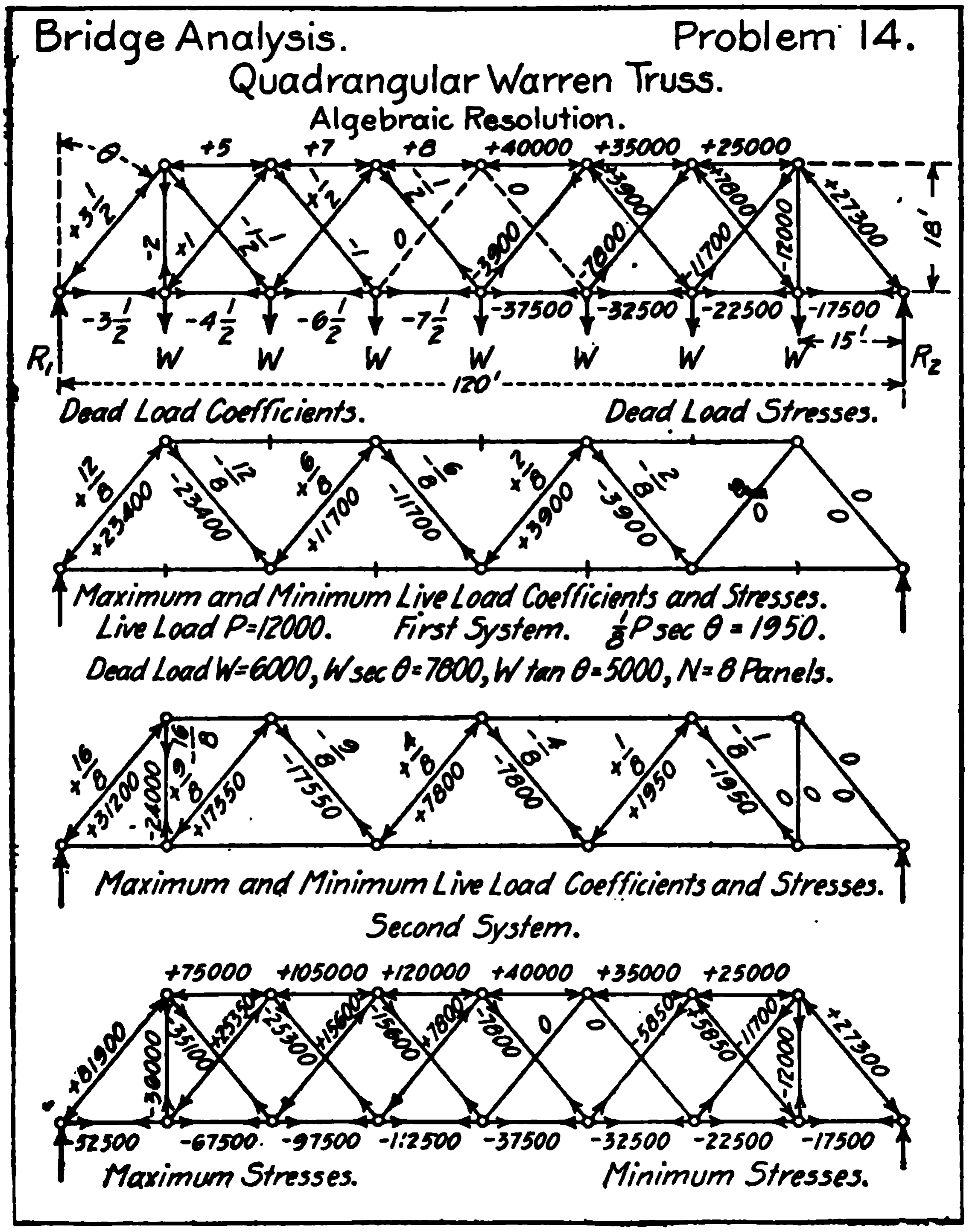
(*a*) **Problem.**—Given a quadrangular Warren truss, span 120′ 0″, panel length 15′ 0″, depth 18′ 0″, dead load 400 lbs. per lineal foot per truss, live load 800 lbs. per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads by algebraic resolution. Scale of truss, 1″ = 20′ 0″.

(*b*) **Methods.**—Construct four truss diagrams as shown.

Dead Load Stresses.—The left reaction is $3\frac{1}{2}W$ and the stresses in the end-post and end panel in the bottom chord are $+ 3\frac{1}{2}W\cdot\sec\theta$ and $- 3\frac{1}{2}W\cdot\tan\theta$, respectively. The other stresses are indeterminate in working from the abutment, and it is necessary to pass to the center of the truss. The two diagonals meeting at the middle of the top chord will have no stress for dead load, and the load at the middle of the truss will be equally divided between the two diagonals meeting at the center of the bottom chord. Passing to the left, the first load to the left of the center is carried directly to the left abutment as shown, while the corresponding load on the right of the center is carried directly to the right abutment. The remaining shears can now be calculated. The chord stresses can now be calculated as in the case of a single intersection truss.

Live Load Stresses.—The coefficients of the maximum chord stresses are the same as the dead load chord coefficients. The maximum and minimum live load web stresses are not statically determinate, but the following solution gives approximate results: Divide the truss into two systems, the first carrying three full loads and the second carrying four full loads as shown. Then calculate the maximum and minimum web stresses in the two systems separately in the usual manner. The maximum web stresses will occur with the longer segment loaded, while the minimum web stresses will occur with the shorter segment loaded. The maximum and minimum stresses are given in the fourth diagram.

(*c*) **Results.**—The live load web stresses as calculated above are approximate, but are on the safe side. The exact solution can be made only by an application of the "Theory of Least Work." This type of truss, with riveted connections, is used by the American Bridge Company for spans of 80 to 152 feet.



PROBLEM 14A. MAXIMUM AND MINIMUM STRESSES IN A QUADRANGULAR WARREN TRUSS BY ALGEBRAIC RESOLUTION.

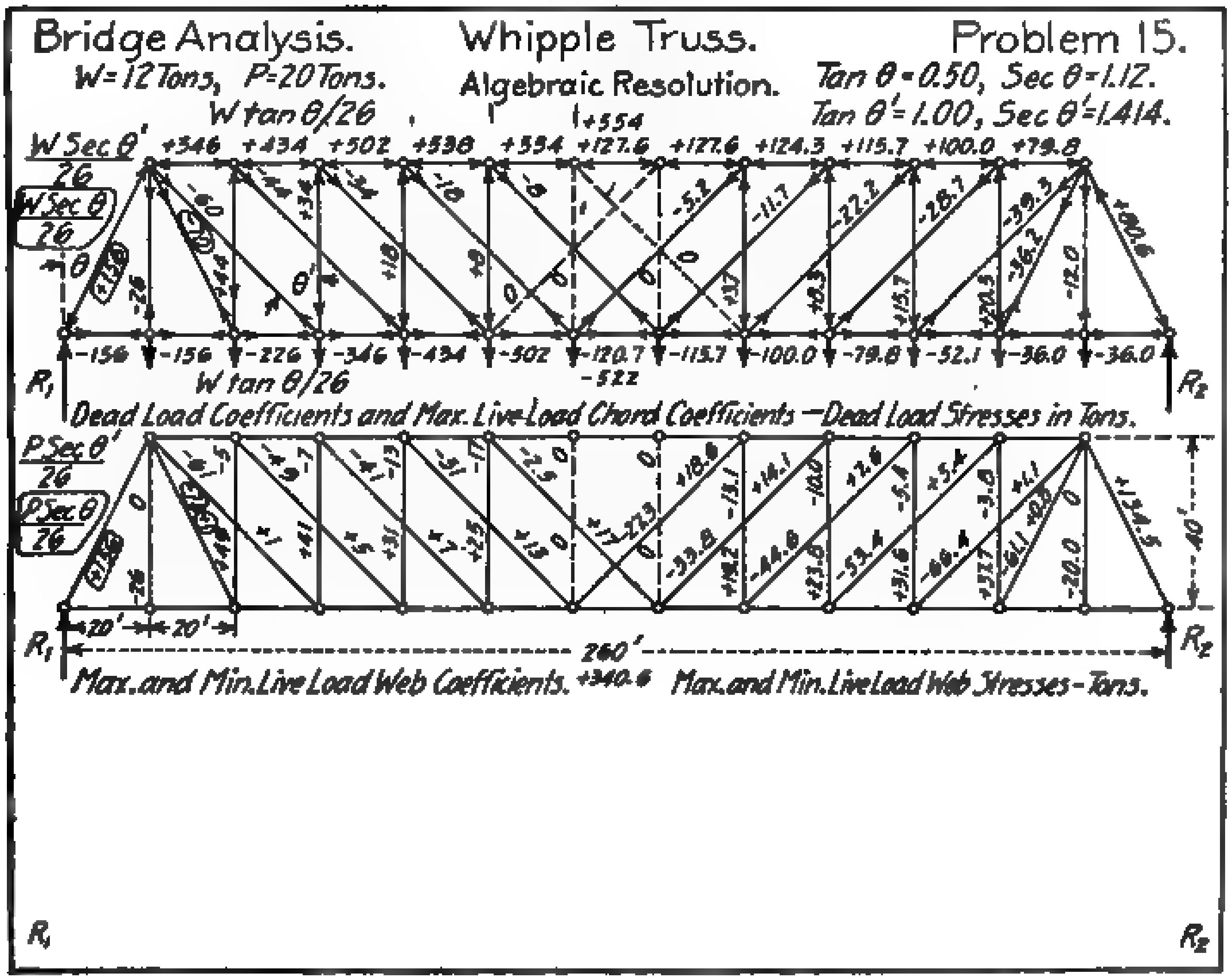
(*a*) **Problem.**—Given a quadrangular Warren truss, span 135′ 0″, panel length 15′ 0″, depth 20′ 0″, dead load 400 lbs. per lineal foot per truss, live load 700 lbs. per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads by algebraic resolution. Scale of truss, 1″ = 20′ 0″.

Problem 15. Maximum and Minimum Stresses in a Whipple Truss by Algebraic Resolution.

(*a*) **Problem.**—Given a Whipple truss, span 260′ 0″, panel length 20′ 0″, depth 40′ 0″, dead load 1,200 lbs. per lineal foot per truss, live load 2,000 lbs. per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads by algebraic resolution. Scale of truss, 1″ = 30′ 0″.

(*b*) **Methods.**—The dead load stresses and the maximum live load chord stresses can be calculated by beginning at the center and calculating the shears, and then calculating the chord stresses as in Problem 14. The maximum and minimum live load web stresses are statically indeterminate as were the web stresses in Problem 14. The usual solution of this problem is to divide the truss into two trusses of single intersection. The dead and the live load chord stresses and the maximum and minimum web stresses are then calculated as for independent trusses. The loads at the foot of the hip verticals are assumed as equally divided between the two systems. The final chord stresses are the sums of the chord stresses in the separate trusses. The stresses in the web members, except the hip vertical, are as given in the separate trusses. In solving the problem the partial truss diagrams should be drawn. The trusses will be unsymmetrical, one being the same as the other turned end for end. With the joints all loaded the dead load chord and web coefficients, and the live load chord coefficients are calculated. In calculating the maximum live load web coefficients the loads are moved off to the right, and the maximum stresses in the webs on the left of the center will occur when the longer segment is loaded, and the minimum stresses in the webs on the right will occur when the shorter segment is loaded. Then with all joints in the truss loaded move the loads off to the left, calculating the maximum web coefficients on the right of the center and the minimum web coefficients on the left of the center. In calculating the stresses from the shears it will be seen that functions of two angles are used. The relation between the two angles is $\tan\theta' = 2\tan\theta$. Web coefficients in terms of θ are enclosed in a ring. The calculation of the chord coefficients may be illustrated by calculating the coefficient of the end panel of the upper chord $= -[-156/26 - 70/26 - 2(60/26)]W\cdot\tan\theta = 346/26W\cdot\tan\theta$.

(*c*) **Results.**—The chord stresses calculated as above do not agree with those calculated by beginning at the center of the truss as in Problem 14. The student should calculate the dead load chord and web stresses and the live load chord stresses as in Problem 14. Whipple trusses were usually built with an odd number of panels. The Whipple truss was formerly quite generally used for long span highway and railway bridges, but is now rarely built, being replaced by the Petit truss.



PROBLEM 15A. MAXIMUM AND MINIMUM STRESSES IN A WHIPPLE TRUSS BY ALGEBRAIC RESOLUTION.

(*a*) **Problem.**—Given a Whipple truss, span 260′ 0″, panel length 20′ 0″, depth 40′ 0″, dead load 1,200 lbs. per lineal foot per truss, live load 2,000 lbs. per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads. Calculate the dead load chord and web stresses and the live load chord stresses as in Problem 14. Scale of truss, 1″ = 30′ 0″.

PROBLEM 16. MAXIMUM AND MINIMUM STRESSES IN A THROUGH BALTIMORE TRUSS BY ALGEBRAIC RESOLUTION.

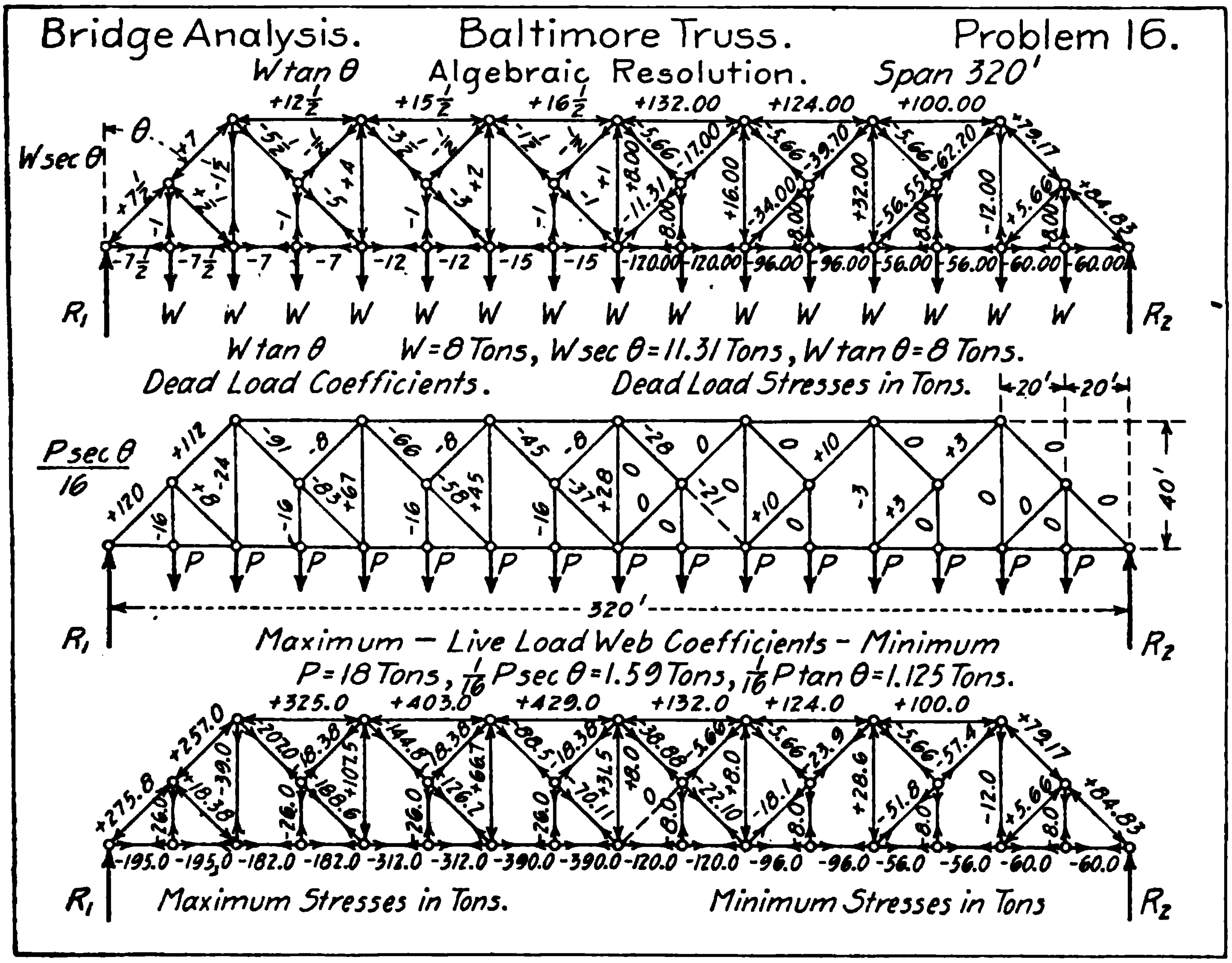
(*a*) **Problem.**—Given a through Baltimore truss, span 320′ 0″, panel length 20′ 0″, depth 40′ 0″, dead load 800 lbs. per lineal foot per truss, live load 1,800 lbs. per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads by algebraic resolution. Scale of truss, 1″ = 40′ 0″.

(*b*) **Methods.**—Construct three truss diagrams as shown.

Dead Load Stresses.—The shear in each of the hangers is W, while the stress in each of the diagonal auxiliary members is $-\frac{1}{2}W\cdot\sec\theta$. The stress in the upper part of the end-post is $(+6\frac{1}{2}+\frac{1}{2})W\cdot\sec\theta = +7W\cdot\sec\theta$, where $+6\frac{1}{2}W\cdot\sec\theta$ is the stress due to the shear and $+\frac{1}{2}W\cdot\sec\theta$ is the stress due to the half load carried toward the center by the auxiliary diagonal member. The stress in the main diagonal in the third panel is $-5\frac{1}{2}W\cdot\sec\theta$, where $5\frac{1}{2}W$ is the shear in the panel; while the stress in the diagonal in the fourth panel is $(-4\frac{1}{2}-\frac{1}{2})W\cdot\sec\theta = -5W\cdot\sec\theta$, where $4\frac{1}{2}W\cdot\sec\theta$ is the stress due to the shear in the panel and $\frac{1}{2}W\cdot\sec\theta$ in the stress carried toward the center of the truss by the auxiliary member. The chord coefficients are calculated as in Problem 13.

Live Load Stresses.—The maximum shear in the third panel occurs with 13 loads to the right of the panel and with no loads to the left of the panel. The shear in the panel is then equal to the left reaction, equals $13\times\frac{1}{2}(13+1)\times P/16 = \frac{91}{16}P$. The stress in the main diagonal in the third panel is then equal to $-\frac{91}{16}P\cdot\sec\theta$. The stress in the main diagonal in the fourth panel is $(-\frac{91}{16}P+\frac{8}{16}P)\sec\theta = -\frac{83}{16}P \sec\theta$, = a maximum, the maximum shear in the panel being $12\times\frac{1}{2}(12+1)\times P/16 = \frac{78}{16}P$. In like manner the maximum stresses are found in 5th and 6th panels when there is a maximum shear in the 5th panel, and in the 7th and 8th panels when there is a maximum shear in the 7th panel. Minimum stresses in the 3d and 4th panels from the right abutment occur when there is a minimum shear in the 3d panel; and in the 5th and 6th panels when there is a minimum shear in the 5th panel.

(*c*) **Results.**—The double panels next to the center require counters. It should be noticed that in calculating the stresses in these coun-



ters the diagonal auxiliary ties will have the dead load stress of + 5.66 tons as a minimum.

Problem 16a. Maximum and Minimum Stresses in a Through Baltimore Truss by Algebraic Resolution.

(*a*) **Problem.**—Given a through Baltimore truss, span 320′ 0″, panel length 20′ 0″, depth 45′ 0″, dead load 800 lbs. per lineal foot per truss, live load 1,800 lbs. per lineal foot per truss. All the auxiliary ties are to be in compression as in the 1st and 2d panels in Problem 16 and as in Problem 6. Calculate the maximum and minimum stresses due to dead and live loads by algebraic resolution. Scale of truss, 1″ = 40′ 0″.

Problem 17. Maximum and Minimum Stresses in a Camel-back Truss by Algebraic Moments.

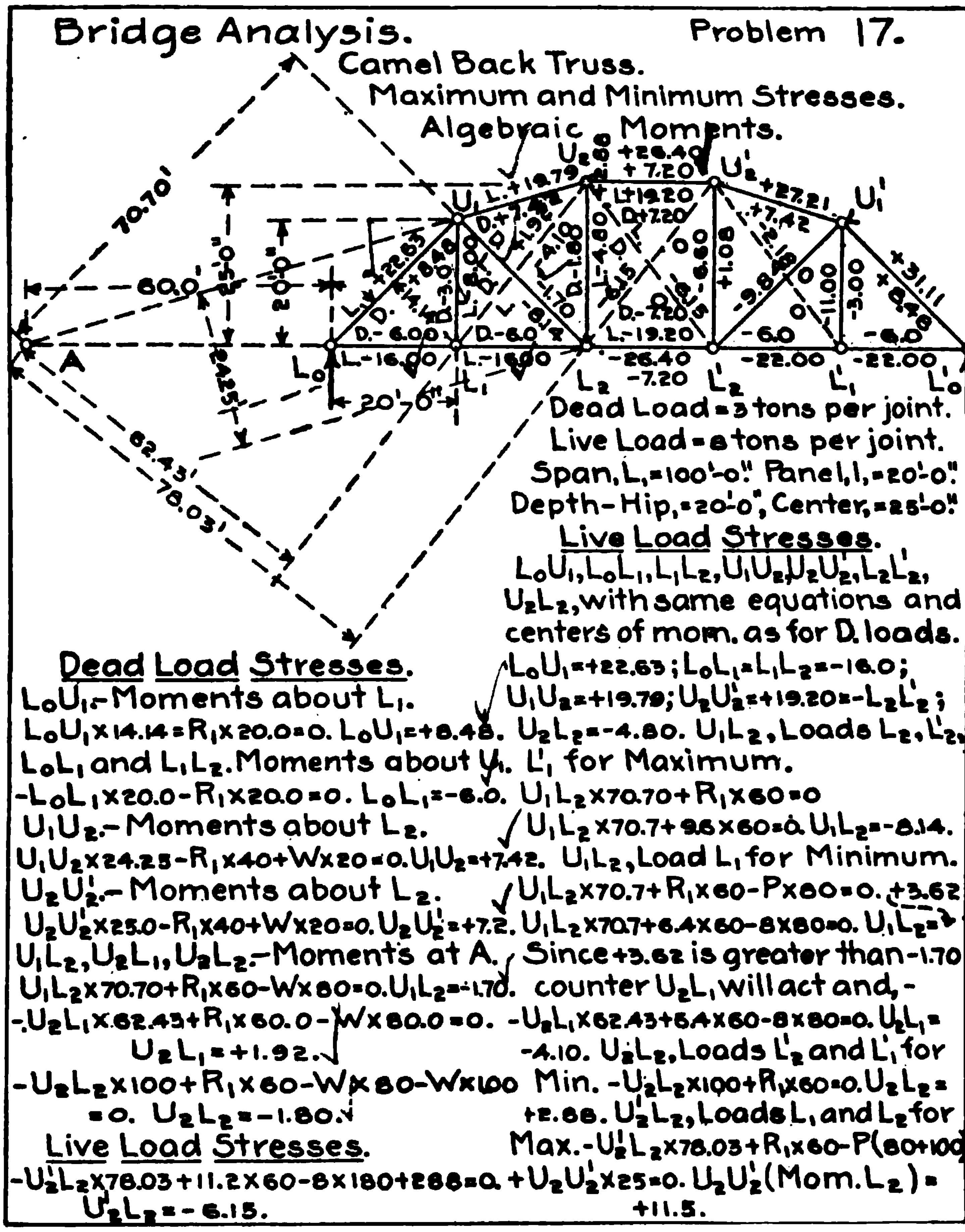
(*a*) **Problem.**—Given a Camel-back truss, span 100′ 0″, panel length 20′ 0″, depth at hip 20′ 0″, depth at center 25′ 0″, dead load 300 lbs. per lineal foot per truss, live load 800 lbs. per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads by algebraic moments. Scale of truss, 1″ = 20′ 0″.

(*b*) *Methods.*—Calculate the arms of the forces as shown and check the values by scaling from the drawing.

Dead Load Stresses.—To calculate the stress in the end-post L_0U_1, take center of moments at L_1, and pass a section cutting L_0U_1, U_1L_1 and L_1L_2, and cutting away the truss to the right. Then assume stress L_0U_1 as an external force acting from the outside toward the cut section, and stress $L_0U_1 \times 14.14 - R_1 \times 20 = 0$. Now $R_1 = 6$ tons and stress $L_0U_1 = +8.48$ tons. To calculate the stresses in L_0L_1 and L_1L_2 take the center of moments at U_1, and pass a section cutting members U_1U_2, U_1L_2 and L_1L_2, and cutting away the truss to the right. Then assume the stress in L_1L_2 as an external force acting from the outside toward the cut section, and $L_1L_2 - R_1 \times 20 = 0$. Now $R_1 = 6$ tons and the stress in $L_0L_1 = L_1L_2 = -6$ tons. To calculate the stress in U_1U_2 take the center of moments at L_2, and pass a section cutting members U_1U_2, U_2L_2 and L_2L_2', and cutting away the truss to the right. Then assume the stress in L_1U_2 as an external force acting from the outside toward the cut section, and $U_1U_2 \times 24.25 - R_1 \times 40 + W \times 20 = 0$. Now $R_1 = 6$, $W = 3$ tons, and the stress in $U_1U_2 = +7.42$ tons. To calculate the stress in U_1L_2 take the center of moments at *A*, and pass a section cutting members U_1U_2, U_1L_2, and L_1L_2, and cutting away the truss to the right. Then assume the stress in U_1L_2 as an external force acting from the outside toward the cut section, and $U_1L_2 \times 70.7 + R_1 \times 60 - W \times 80 = 0$. Now $R_1 = 6$ tons and $W = 3$ tons, and $U_1L_2 \times 70.7 = -120$ ft.-tons, and stress $U_1L_2 = -1.70$ tons. The other dead load stresses are calculated as shown.

Live Load Stresses.—The live load chord stresses are equal to the dead load chord stresses multiplied by 8/3. The maximum stress in U_1L_2 will occur with loads at L_2, L_2', and L_1', while the maximum stress in counter U_2L_1 will occur with a load at L_1 only. The maximum tension in U_2L_2 will occur with all the live loads on the bridge, while the maximum compression will occur when there is a maximum stress in the counter U_2L_2', loads at L_2' and L_1'. The details of the solution are shown in the problem.

(*c*) **Results.**—The stress in the counter U_2L_2' and the chords U_2U_2' and L_2L_2' may be calculated by the method of coefficients, and will be the same as for a truss with parallel chords having a depth of 25′ 0″. The maximum stress in U_2L_2 will occur with loads L_2' and L_1' on the bridge, when the left reaction equals $2 \times 3P/5 = 6/5P$. The stress in $U_2L_2 = -6/5P \cdot \sec\theta = -6.15$ tons.



PROBLEM 17A. MAXIMUM AND MINIMUM STRESSES IN A CAMEL-BACK TRUSS BY ALGEBRAIC MOMENTS.

(*a*) **Problem.**—Given a Camel-back truss, span 120′ 0″, panel length 20′ 0″, depth at hip 25′ 0″, depth at U_2 30′ 0″, depth at U_3 30′ 0″, dead load 300 lbs. per lineal foot per truss, live load 800 lbs. per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads.

PROBLEM 18. MAXIMUM AND MINIMUM STRESSES IN A THROUGH WARREN TRUSS BY GRAPHIC MOMENTS.

(*a*) **Problem.**—Given a through Warren truss, span 140′ 0″, panel length 20′ 0″, depth 20′ 0″, dead load 800 lbs. per lineal foot per truss, live load 1,200 lbs. per lineal foot per truss. Calculate the maximum and minimum stresses by graphic moments. Scale of truss, 1″ = 20′ 0″. Scale of loads, 1″ = 50,000 lbs.

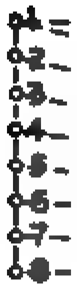
(*b*) **Methods.** *Chord Stresses.*—Calculate the center ordinate of the parabola $= w \cdot L^2/8d = 98{,}000$ lbs., and lay it off at 5 to the prescribed scale. Now lay off the vertical line 1–5 at the left and right abutments. Make 1–2 = 2–3 = 3–4 = 2 (4–5). Draw the inclined lines 1–5, 2–5, 3–5, 4–5, 5–5. The intersections of these lines with verticals let drop from the lower chord points are points in the stress parabola for the upper chord stresses. The stresses in the lower chords are the arithmetical means of the stresses in the upper chords on each side. By changing the scale the live load stresses may be scaled directly from the diagram.

Web Stresses.—At the distance of a panel to the left of the left abutment lay off the vertical line 1–8 equal to one-half the total live load on the truss, to the prescribed scale, equal $1{,}200 \times 70 = 84{,}000$ lbs. Now divide the line 1–8 into as many equal parts as there are panels in the truss, and mark the points of division 2, 3, 4, etc. Connect these points of division with the panel point 7, the first panel point to the left of the right abutment. Drop verticals from the panel points of the lower chord of the truss to the line 1–8, and the intersections of like numbered lines will give points on the curve of maximum live load shears.

To construct the dead load shear diagram, lay off $3W$, downward to the prescribed scale under the left abutment, and reduce the shear under each load to the right by W, until the dead load shear is $-3W$ at the right abutment. The dead load shear diagram is then constructed as shown.

Maximum and Minimum Web Stresses.—The maximum shear in any panel is then the ordinate to the right of the panel point on the left end of the panel, and the stresses in the web members are calculated by drawing lines parallel to the corresponding member as shown. Positive stresses are measured downwards from the live load shear curve, and negative stresses are measured upwards from the live load shear curve.

(*c*) **Results.**—This method is an excellent one for illustrating the effect of the different systems of loads, but consumes too much time to be of practical use. It should be noted that the maximum ordinate to the chord parabola is not a chord stress in a Warren truss with an odd number of panels.



Problem 18a. Maximum and Minimum Stresses in a Through Warren Truss by Graphic Moments.

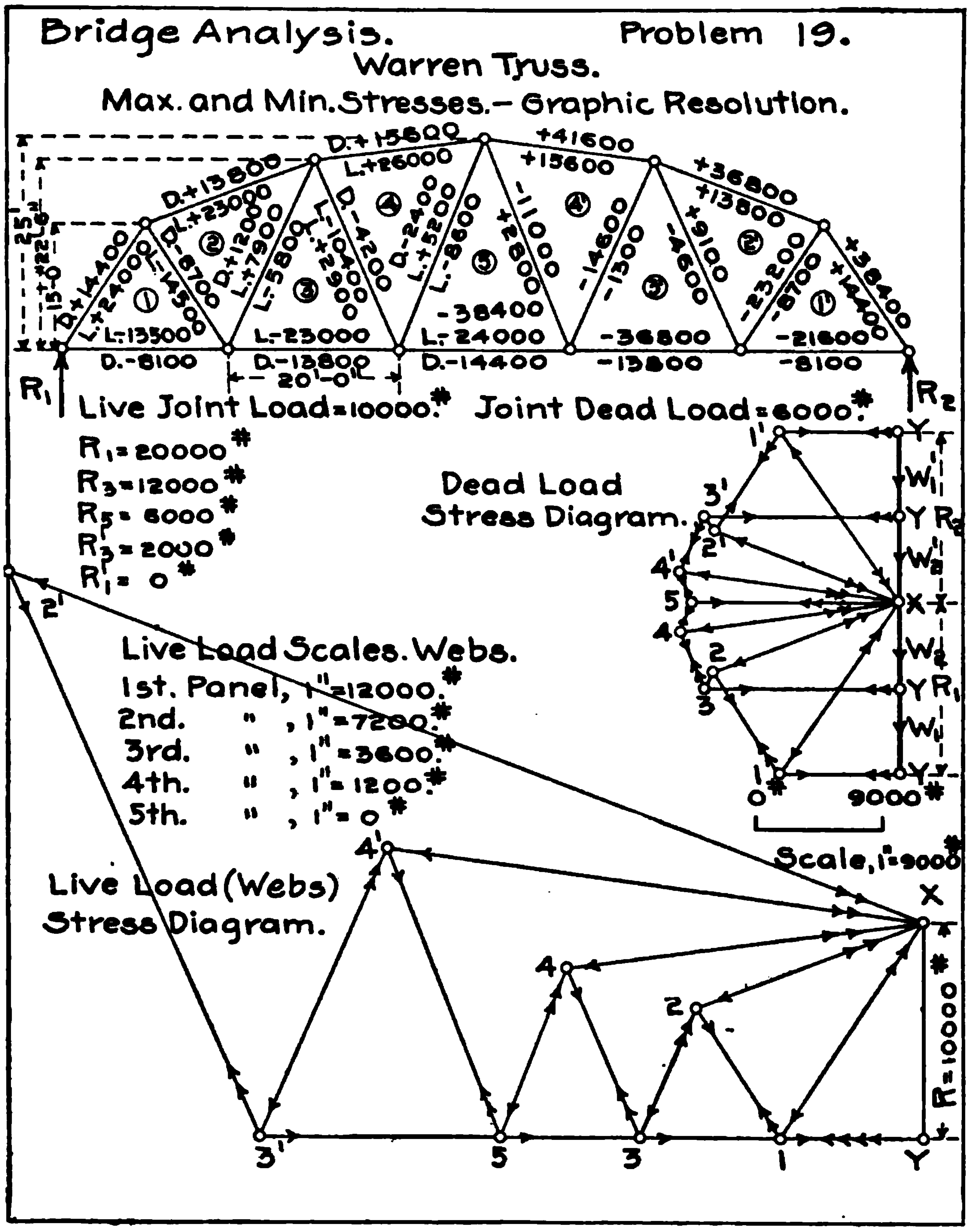
(*a*) **Problem.**—Given a through Warren truss, span 160′ 0″, panel length 20′ 0″, depth 24′ 0″, dead load 900 lbs. per lineal foot per truss, live load 1,200 lbs. per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads by graphic moments. Scale of truss, 1″ = 25′ 0″. Scale of loads, 1″ = 50,000 lbs.

PROBLEM 19. MAXIMUM AND MINIMUM STRESSES IN AN INCLINED CHORD THROUGH WARREN TRUSS BY GRAPHIC RESOLUTION.

(*a*) **Problem.**—Given an inclined chord through Warren truss, span 100′ 0″, panel length 20′ 0″, depth at the hip 15′ 0″, depth at the second panel 22′ 6″, depth at the center 25′ 0″, dead load 600 lbs. per lineal foot per truss, live load 1,000 lbs. per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads by graphic resolution. Scale of truss, 1″ = 15′ 0″. Scale of dead loads, 1″ = 9,000 lbs. Scale of live loads as shown.

(*b*) **Methods.**—Construct a truss diagram and calculate the dead load stresses in the usual way as shown. The live load chord stresses are found by multiplying the dead load chord stresses by 5/3. To calculate the maximum and minimum web stresses proceed as follows: Assume that the truss is fixed at the right abutment and that the left reaction is R_1 = say 10,000 lbs. with no loads on the bridge. Then beginning at the left reaction R_1, calculate by graphic resolution the stresses in the different members of the truss due to the left reaction of 10,000 lbs., there being no loads on the bridge. The reaction is laid off to a scale of 1″ = 6,000 lbs. Now to calculate the maximum live load stress in any web member multiply the stress as scaled from the diagram by the ratio of the left reaction which produces the maximum stress to 10,000 lbs. For example, the member 1–2 has a maximum stress with all the joints loaded and the reaction is 20,000 lbs., or the scale of the stress is 1″ = 12,000 lbs. The stress 1–2 then equals — 14,500 lbs. The maximum live load stress in 2–3 occurs with loads at the three panel points at the right, and $R_3 = \frac{1}{5}(3 \times 2P) = 12{,}000$ lbs., or the scale of the stress in the diagram is 1″ = 7,200 lbs., and the stress in 2–3 equals + 7,900 lbs. The stresses in the remaining web members are calculated in the same manner.

(*c*) **Results.**—This solution may be used to calculate the maximum and minimum stresses in any truss, but it is best adapted to the solution of stresses in trusses like the one shown. The maximum and minimum stresses are given on the right hand side of the truss diagram.



PROBLEM 19A. MAXIMUM AND MINIMUM STRESSES IN AN INCLINED CHORD THROUGH WARREN TRUSS BY GRAPHIC RESOLUTION.

(*a*) **Problem.**—Given an inclined chord through Warren truss, span 120′ 0″, panel length 20′ 0″, depth at the hip 15′ 0″, depth at the second panel in the top chord 22′ 6″, depth at the third panel 25′ 0″ (middle panel has parallel chords), dead load 600 lbs. per lineal foot per truss, live load 1,100 lbs. per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads by graphic resolution. Scale of truss, 1″ = 20′ 0″. Scale of dead loads, 1″ = 10,000 lbs. Scale of live loads as calculated.

PROBLEM 20. MAXIMUM AND MINIMUM STRESSES IN A PETIT TRUSS BY ALGEBRAIC MOMENTS.

(*a*) **Problem.**—Given a Petit truss, span 350′ 0″, panel length 25′ 0″, depth at the hip 50′ 0″, depth at center 58′ 0″, dead load 0.9 tons per lineal foot per truss, live load 1.4 tons per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads by algebraic moments. Scale of truss, 1″ = 40′ 0″. Scale of lever arms, any convenient scale.

(*b*) **Methods.**—Construct a truss diagram carefully to scale as shown. Construct one-half the truss to scale on a large piece of paper and calculate the lever arms as shown, and check by scaling from the diagram. The methods of calculation will be shown by two examples:

1. *Stresses in Tie 6–7. Dead Load Stress.*—Pass a section cutting members 7–X, 6–7, and 6–Y, and cutting away the truss to the right. The center of moments will be at A, the intersection of chords 7–X and 6–Y. Now assume the stress in 6–7 as an external force acting from the outside toward the cut section. Then for equilibrium 6–7 $\times 477.0 + R_1 \times 575 - 3W \times 625 = 0$. Now $R_1 = 146.25$ tons and $W = 22.5$ tons, and solving the equation gives stress 6–7 = -87.8 tons.

Live Load Stresses.—The maximum live load stress in 6–7 will occur with the longer segment of the truss loaded. Taking moments about point A as for the dead loads the maximum live load stress 6–7 $\times 477.0 + R_1 \times 575 = 0$. Now $R_1 = 55/14 \times 35$ tons $= 137.5$ tons, and the stress in 6–7 = -165.8 tons.

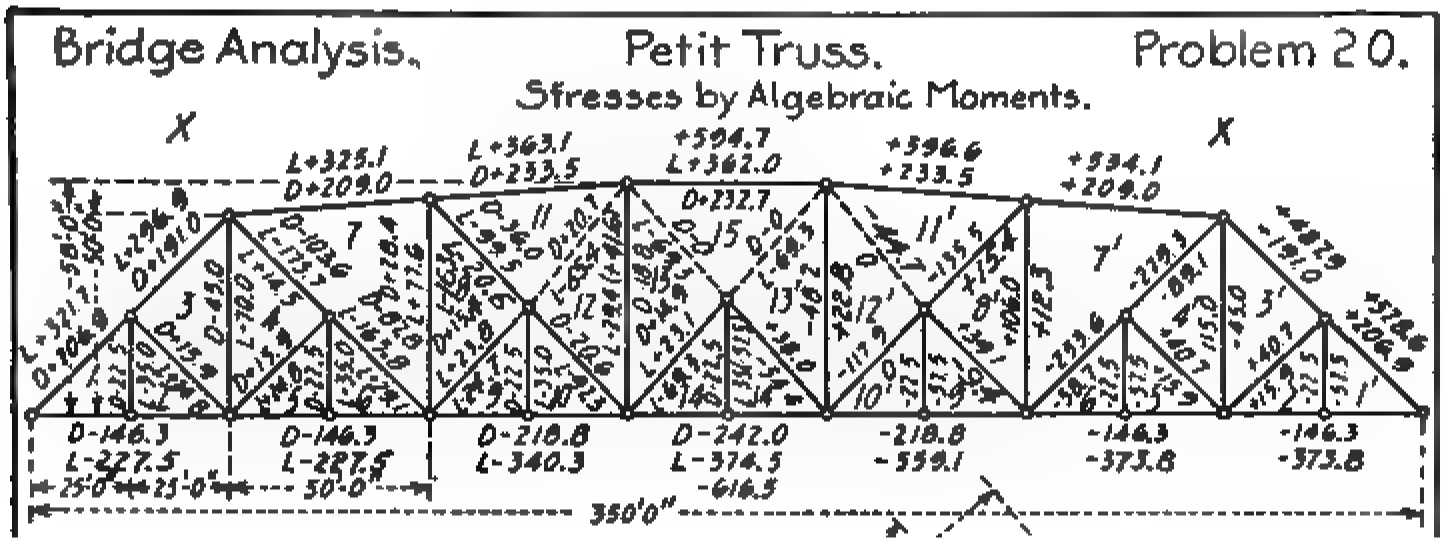
The minimum live load stress in 6–7 will occur with the shorter segment of the truss loaded. Taking moments about the point A, 6–7 $\times 477.0 + R_1 \times 575 - 3P \times 625 = 0$. Now $R_1 = 90$ tons, $P = 35$ tons, and stress in 6–7 = $+29.1$ tons.

2. *Stresses in Tie 4–7. Dead Load Stress.*—Pass a section cutting members 7–X, 4–7, 4–5 and 5–Y, and cutting away the truss to the right. Now assume the stress in 4–7 as an external force acting from the outside toward the cut section. Then for equilibrium about the point A, stress 4–7 $\times 477.0 + R_1 \times 575 -$ stress 4–5 $\times 442.0 - 2W \times 612.5 = 0$. Now the number 4–5 will carry one-half the load carried by 5–6, and the stress equals $1/2 \times 22.5 \times 1.414 = +15.9$ tons. $R_1 = 146.25$ tons, and $2W = 45$ tons. Then stress 4–7 = -103.6 tons.

Live Load Stresses.—The maximum live load stress in 4–7 will occur with the longer segment loaded. Taking moments about A as for dead loads, stress 4–7 $\times 477.0 + R_1 \times 575 -$ stress 4–5 $\times 442.0 = 0$. Now stress 4–5 = $+24.8$ tons, and $R_1 = 66/14 \times 35 = 165$ tons. Then stress 4–7 = -175.7 tons.

The minimum live load stress in 4–7 will occur with two loads to the left of the panel. Taking moments about the point A, the stress 4–7 $\times 477.0 + R_1 \times 575 - 2P \times 612.5 = 0$. Now $R_1 = 62.5$ tons and $2P = 70$ tons. Then stress 4–7 = $+14.5$ tons.

The stresses in the members in the first and second panels and in



the two middle panels may be calculated by coefficients. Check up the dead load chord stresses by comparing with the stresses obtained by graphic resolution in Problem 6.

(c) **Results.**—The auxiliary members carry the stresses directly toward the abutments and there is no ambiguity of loading as in the case of a truss subdivided as in Problem 16. However, the method of subdividing shown in Problem 16 is used in preference to that shown in this problem. The Petit truss is quite generally used for long span pin-connected highway and railway bridges.

Problem 20a. Maximum and Minimum Stresses in a Petit Truss by Algebraic Moments.

(*a*) **Problem.**—Given a Petit truss with the same span and loads as in Problem 20, the auxiliary bracing to be the same as in Problem 16. Calculate the maximum and minimum stresses due to dead and live loads by algebraic moments.

PROBLEM 21. LIVE LOAD STRESSES IN A THROUGH PRATT TRUSS FOR COOPER'S E 40 LOADING.

(*a*) **Problem.**—Given a Pratt truss, span 165′ 0″, panel length 23′ 6¾″, depth 30′ 0″, live load Cooper's E 40 loading. Calculate the position of the loads and the maximum and minimum stresses due to the prescribed loading by algebraic moments. Scale of truss, 1″ = 25′ 0″.

(*b*) **Methods.** *Chord Stresses.*—Calculate the position of the wheels for a maximum bending moment at the different joints in the lower chord. The criterion for maximum bending moment at any joint in a Pratt truss is, "the average load on the left of the section must be the same as the average load on the entire bridge." Having determined the wheel that is at the joint for a maximum moment, calculate the maximum bending moment as shown. Having calculated the maximum bending moments, the chord stresses are found by dividing the bending moment by the depth of the truss. The moment diagram is given in Fig. 95.

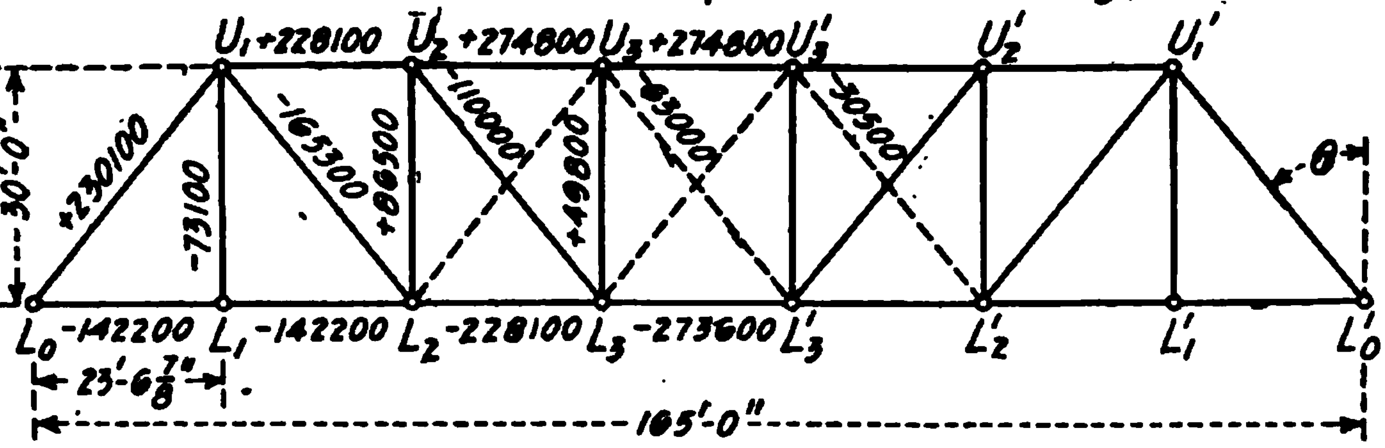
Web Stresses.—Calculate the position of the wheels for maximum shears in the different panels. The criterion for maximum shear in a panel is, "the load on the panel must equal the load on the bridge divided by the number of panels." The criterion for maximum bending moment at L_1 is the same as the criterion for maximum shear in panel L_0L_1. Having determined the position of the wheels for maximum shears in the different panels, calculate the maximum shears as shown. The stress in a web is equal to the shear in the panel multiplied by $\sec\theta$.

Floorbeam Reaction.—The stress in the hip vertical U_1L_1 is equal to the maximum floorbeam reaction. This is calculated as follows: Take a simple beam with a span equal to the sum of two panel lengths and calculate the maximum bending moment at the point in the beam corresponding to the panel point; in this case it will be the center of the span. This bending moment multiplied by the sum of the panel lengths divided by the product of the panel lengths will be the maximum floorbeam reaction; in this case the maximum bending moment at the center will be multiplied by 2 divided by the panel length.

(*c*) **Results.**—When the maximum stresses occur in chords U_2U_3, U_3U_3' and L_3L_3', counter $U_3'L_3$ is in action. It occasionally happens that there is more than one position of the loading that will satisfy the criterion for maximum bending moment. In this case the moments for each case must be calculated. The equivalent uniform load for the truss from Fig. 48, is 2,475 lbs. per lineal foot per truss. This load is used for calculating maximum moments and shears, using equal joint loads in the same manner as for highway bridges. To calculate the maximum floorbeam reaction an equivalent load of 3,110 lbs. per lineal foot is used. The stringers are always designed for actual wheel concentrations, or by means of tables calculated for actual wheel loads. For equivalent uniform loads for floorbeams, stringers, etc., see Table X.

Bridge Analysis Problem 21.

Stresses for Cooper's E40 Loading.



Chord Stresses:—

L_1, Maximum Moment at.—Try wheel 4 at L_1. Total Load on Bridge = 284 + 50.43 × 2 = 384.86. Average panel load = 384.86 ÷ 7 = 54.98. Load on left of L_1 = 50 or 70. Wheel 4 gives maximum moment at L_1.

L_2, Maximum Moment at.—Try wheel 7 at L_2. Total Load on Bridge = 284 ± 45.86 × 2 = 375.72. Average panel load = 375.72 ÷ 7 = 53.67. Average panel load on left of L_2 = 103/2 or 116/2 = 51.5 or 58. Wheel 7 gives maximum moment at L_2. The other loadings are found in like manner.

Maximum Moments and Chord Stresses

Center of Moments	Wheel giving Max. Mom.	Maximum Moment, Thousand Foot-Pounds	Stresses in Chords Max. Mom. ÷ 30	
			Member	Stress Lbs.
L_1	4	$(16364+284\times50.43+50.43^2)\,1/7-480=4267$	L_0L_1 & L_1L_2	142200
L_2	7	$(16364+284\times45.86+45.86^2)\,2/7-2155=6843$	U_1U_2 & L_2L_3	228100
L_3	11	$(16364+284\times49.29+49.29^2)\,3/7-5848=8244$	U_2U_3 & U_3U_3'	274800
L_3'	13	$(16364+284\times35.71+35.71^2)\,4/7-7668=8207$	L_3L_3'	273600

Web Stresses:—

Panel L_0L_1. Maximum Shear. Wheel 4 at L_1 gives maximum shear.

Panel L_1L_2. Maximum Shear. Try wheel 3 at L_2. Average load on bridge = [(284 + 117.86 − 96) × 2] 1/7 = 46.82. Load on panel = 30 or 50. Wheel 3 at L_2 gives maximum shear in panel L_1L_2. The other loadings are found in like manner.

Maximum Shears and Web Stresses

Panel	Wheel giving Max. Shear	Maximum Shear, Thousand Pounds	Stresses in Webs Shear × Sec θ (1.272)	
			Member	Stress Lbs.
L_0L_1	4 at L_1	$(16364+284\times50.43+50.43^2)\div165-480\div23.57=180.93$	L_0U_1	230100
L_1L_2	3 at L_2	$(16364+284\times21.86+21.86^2)\div165-230\div23.57=129.94$	U_1L_2	165300
L_2L_3	3 at L_3	$(16364+284\times1.72)\div165-230\div23.57=86.46$	U_2L_3	110000
L_3L_3'	3 at L_3'	$(8728+232\times4.71)\div165-230\div23.57=49.76$	U_3L_3'	63000
$L_3'L_2'$	2 at L_2'	$(4632+142\times7.14)\div165-80\div23.57=23.94$	$U_3'L_2'$	30500

Stress U_2L_2 = Shear in panel L_2L_3 = 86500; Stress U_3L_3 = Shear in panel L_3L_3' = 49800
Stress in U_1L_1 = Maximum floorbeam reaction = 73100 lbs.

PROBLEM 21A. LIVE LOAD STRESSES IN A THROUGH PRATT TRUSS FOR COOPER'S E 40 LOADING.

(*a*) **Problem.**—Given a Pratt truss, span 200′ 0″, panel length 25′ 0″, depth 32′ 0″, live load Cooper's E 40 loading. Calculate the position of the loads and the maximum and minimum stresses due to the prescribed loading by algebraic moments. Check the concentrated live load stresses by calculating the maximum and minimum stresses for the equivalent uniform live load as given in Fig. 48. Scale of truss, 1″ = 30′ 0″.

PROBLEM 22. STRESSES IN THE PORTAL OF A BBIDGE BY ALGEBRAIC MOMENTS AND GRAPHIC RESOLUTION.

(*a*) **Problem.**—Given the portal of a bridge of the type shown, inclined height 30′ 0″, center to center width 15′ 0″, load $R=2{,}000$ lbs., end-posts pin-connected at the base. Calculate the stresses by algebraic moments and check by graphic resolution. Scales as shown.

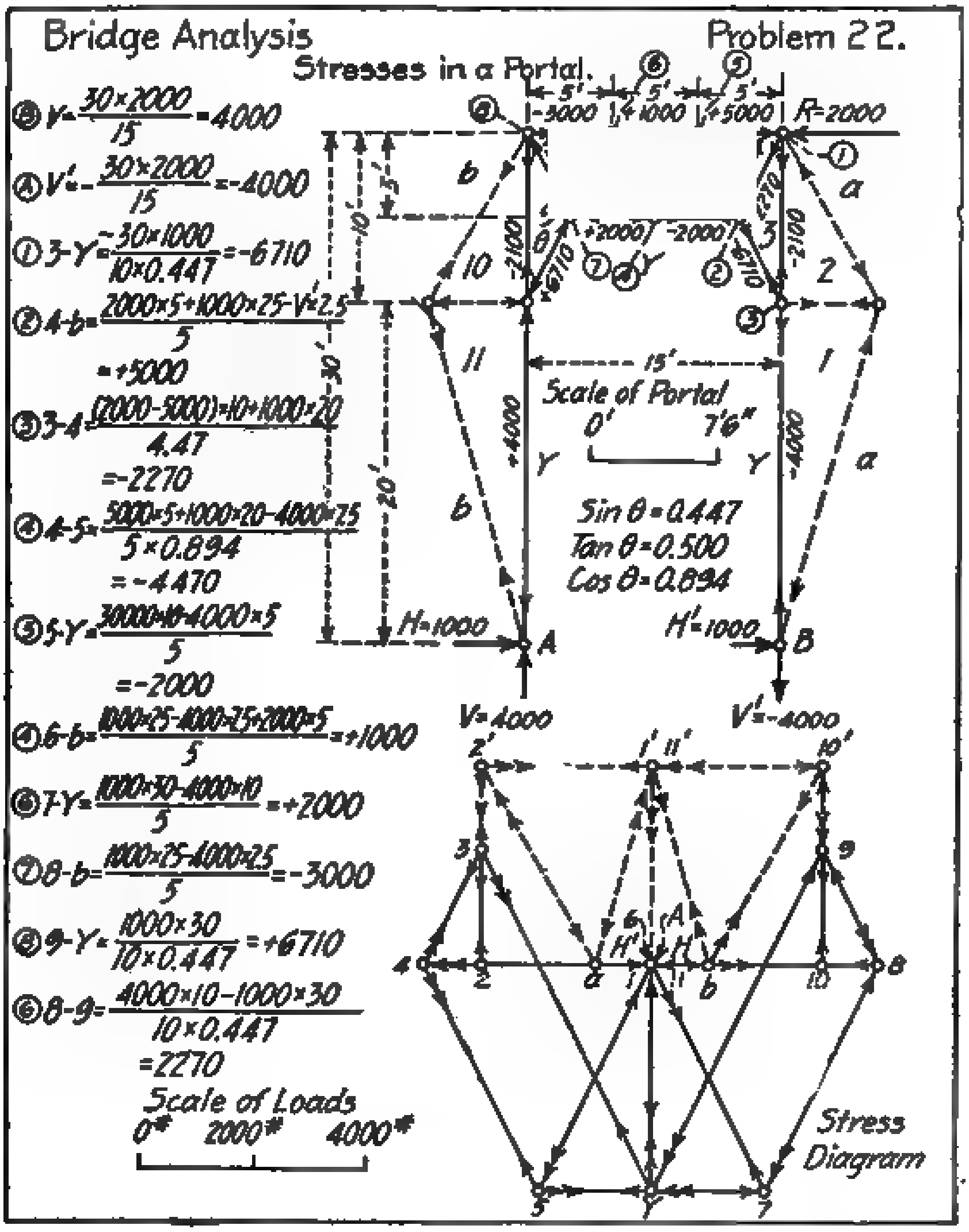
(*b*) **Methods.**—Now $H=H'=1{,}000$ lbs. $V=-V'$, and by taking moments about B, $V=30\times 2{,}000/15=4{,}000$ lbs. $=-V'$.

Algebraic Moments.—In passing sections, care should be used to avoid cutting the end-posts for the reason that these members are subject to bending stresses in addition to the direct stresses. To calculate the stress in member 3–Y take the center of moments at joint (1) and pass a section cutting members 4–b, 3–4 and 3–Y, and cutting the portal away to the left of the section. Then assumes stress 3–Y as an external force acting from the outside toward the cut section, and 3–Y $\times\ 10\times 0.447\sin\theta+H\times 1{,}000=0$. The stress in 3–$Y=-6{,}710$ lbs. The remaining stresses are calculated as shown.

Graphic Resolution.—Lay off a–$A=A$–$b=H=1{,}000$ lbs., and A–$Y=V'=4{,}000$ lbs. Then beginning at point B in the portal the force polygon for equilibrium is a–A–Y–1′–a, in which 1′–a is the stress in the auxiliary member 1–a, and Y–1′ is the stress in the post 1–Y when the auxiliary member is acting. The true stress in 1–Y is equal to the algebraic sum of the vertical components of the stress 1′–a and Y–1′, and equals $V'=-4{,}000$ lbs. Next complete the force triangle at the intersection of the auxiliary members. Stress 1′–a is known and the force triangle is a–1′–2′–a, the forces acting as shown. The stress diagram is carried through in the order shown, checking up at the point A. The correct stresses are shown by the full lines in the stress diagram. The true stress in 3–2 will produce equilibrium for vertical stresses at joint (1) as shown. The maximum shear in the posts is $H=1{,}000$ lbs. The maximum bending moment in the posts will occur at the foot of the member 3–Y, joint (3), and is $M=1{,}000\times 20\times 12=240{,}000$ in.-lbs.

(*c*) **Results.**—The method of graphic resolution requires less work and is more simple than the method of algebraic moments.

Note: The portal is not pin-connected at joints (3) and the corresponding joint on the opposite side, as might be inferred from the figure.



Problem 22a. Stresses in the Portal of a Bridge by Algebraic Moments and Graphic Resolution.

(*a*) **Problem.**—Given the portal shown in Problem 22, except that the posts are fixed at their bases. Calculate the stresses by algebraic moments and check by graphic resolution. Assume the point of contraflexure as half way between the base of the post and the foot of the knee brace. Scales as in Fig. 22.

PROBLEM 23. WIND LOAD STRESSES IN A TRESTLE BENT.

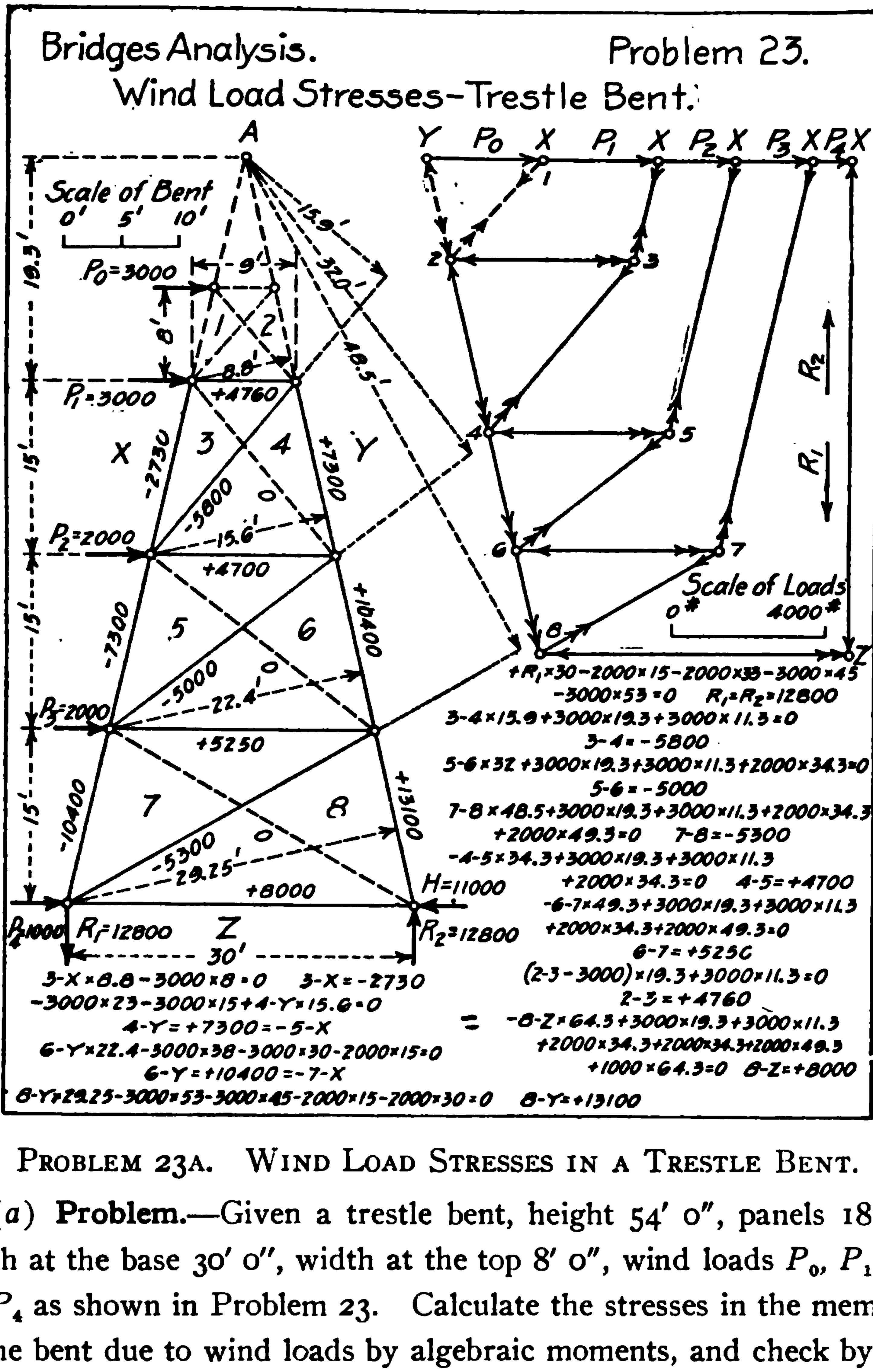
(*a*) **Problem.**—Given a trestle bent, height 45′ 0″, width at the base 30′ 0″, width at the top 9′ 0″, wind loads P_0, P_1, P_2, P_3, P_4, as shown. Calculate the stresses in the members of the bent due to wind loads by algebraic moments, and check by calculating the stresses by graphic resolution. Assume that the diagonal members are tension members, and that the dotted members are not acting for the wind blowing as shown. Scale of truss, 1″ = 10′ 0″. Scale of loads, 1″ = 2,000 lbs.

(*b*) **Methods.**—*Algebraic Moments.*—To calculate the stresses in the diagonal members take centers of moments about the point A, the point of intersection of the inclined posts. Then to calculate the stress in 3–4, pass a section cutting members 3–X, 3–4 and 4–Y; assume that the stress in 3–4 is an external force acting from the outside toward the cut section, and $3\text{–}4 \times 15.9' + 3{,}000 \times 19.3' + 3{,}000 \times 11.3' = 0$. The stress $3\text{–}4 = -5{,}800$ lbs. Stresses in 4–5, 5–6, 6–7, 7–8 and 8–Z are calculated in a similar manner. To obtain reaction R_1 take moments about R_2, and $R_1 \times 30' - 2{,}000 \times 15' - 2{,}000 \times 30' - 3{,}000 \times 45' - 3{,}000 \times 53' = 0$. Then $R_1 = 12{,}800$ lbs. $= -R_2$.

To calculate the stress in 4–Y, take center of moments at joint P_2, and pass a section cutting members 5–X, 4–5 and 4–Y, and assume the stress in 4–Y as an external force acting from the outside toward the cut section. Then $4\text{–}Y \times 15.6' - 3{,}000 \times 15' - 3{,}000 \times 23' = 0$. Then $4\text{–}Y = +7{,}300$ lbs.

Graphic Resolution.—The load P_0 is assumed as transferred to the bent by means of the auxiliary members. The loads P_0, P_1, P_2, P_3, P_4 are laid off as shown, and with the load P_0 the stress triangle Y–X–2 is drawn. The remainder of the solution is easily followed.

(*c*) **Results.**—The stress in the auxiliary member 2–Y acts as a load at the top of post 4–Y. Load P_0 is the wind load on the train and is transferred to the rails by the car. For the reason that the wind may blow from the opposite direction, both sets of stresses must be considered in combination with the dead and live load stresses in designing the columns.



PROBLEM 23A. WIND LOAD STRESSES IN A TRESTLE BENT.

(*a*) **Problem.**—Given a trestle bent, height 54′ 0″, panels 18′ 0″, width at the base 30′ 0″, width at the top 8′ 0″, wind loads P_0, P_1, P_2, P_3, P_4 as shown in Problem 23. Calculate the stresses in the members of the bent due to wind loads by algebraic moments, and check by calculating the stresses by graphic resolution. Assume that the diagonal members are tension members, and that the dotted members are not acting for the wind blowing as shown. Scale of truss, 1″ = 10′ 0″. Scale of loads, 1″ = 2,000 lbs.

Problem 24. Wind Load Stresses in a Transverse Bent by Graphic Resolution.

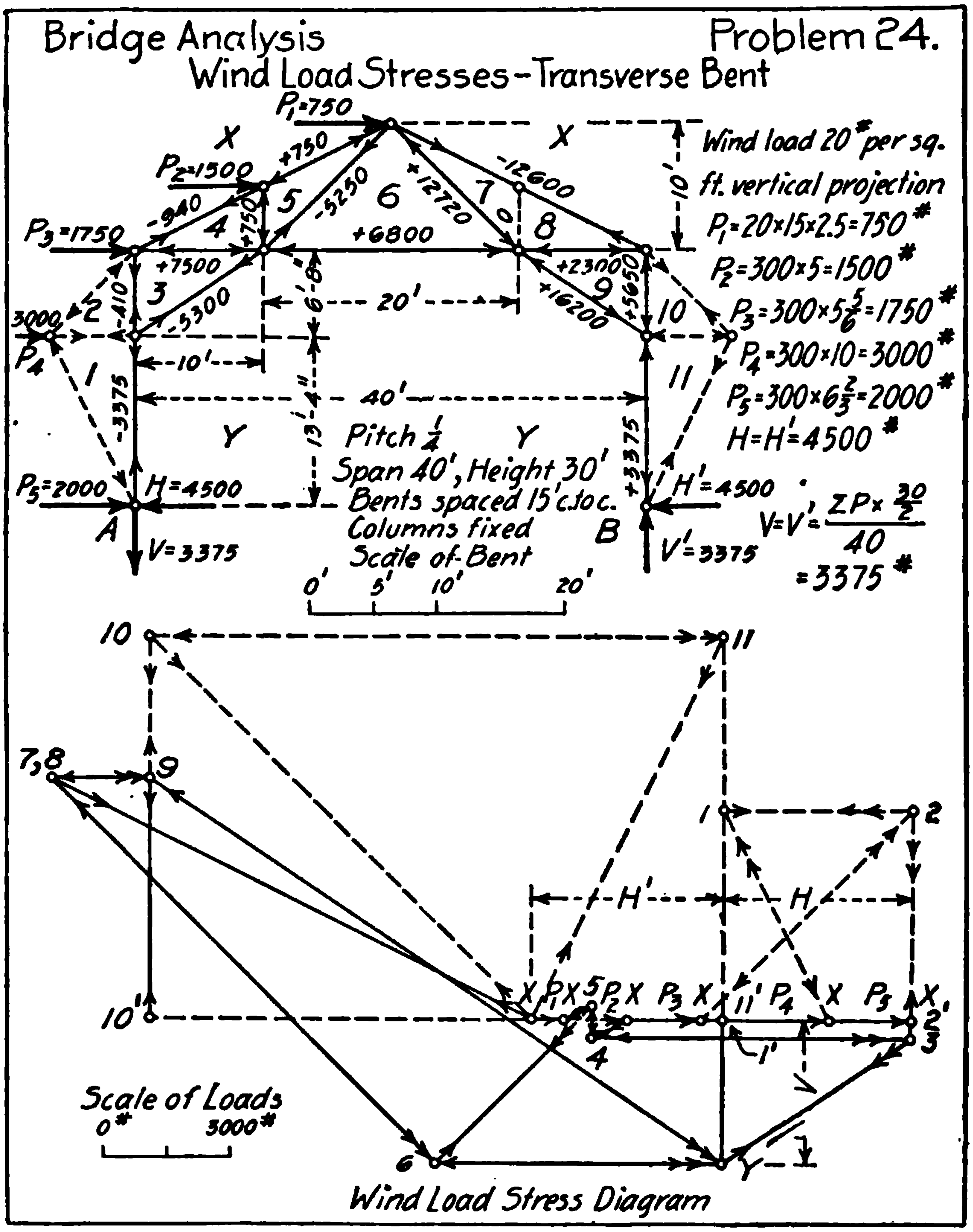
(*a*) **Problem.**—Given a transverse bent, span 40′ 0,″ pitch of roof $\frac{1}{4}$, height of posts 20′ 0″, posts pin-connected at the base, wind load 20 lbs. per square foot of vertical projection. Calculate the wind load stresses in the bent by graphic resolution. Scale of bent, 1″ = 10′ 0″. Scale of loads, 1″ = 3,000 lbs.

(*b*) **Methods.**—Now $H = \frac{1}{2}\Sigma P = 4{,}500$ lbs. $= H'$. To calculate V take moments about the foot of the right-hand post, and $V \times 40' - 3{,}000 \times 13\frac{1}{3}' - 1{,}750 \times 20' - 1{,}500 \times 25' - 750 \times 30' = 0$. Then V $= + 3{,}375$ lbs. $= - V'$.

To construct the stress diagram lay off the load line $P_1 + P_2 + P_3 + P_4 + P_5$, and $1\text{–}Y = V = 3{,}375$ lbs. Beginning at the foot of the windward post, V acts downward, $H = X\text{–}1$ acts to the left, P_5 acts to the right. The polygon is closed by drawing lines parallel to 1–X and 1–Y, the final stress polygon being Y–1–X–X–1′. Then pass to the load P_4 in the transverse bent, and in the stress diagram P_4 acts to the right, 1–X acts upwards to the left, 1–2 acts to the right, and 2–X acts downwards to the left, closing the polygon. The remainder of the stress diagram is drawn in a similar manner, passing to the foot of the knee brace, then to the top of the post, etc., finally checking up at the foot of the leeward post. The maximum shear is in the leeward post, below the knee brace the shear is $H = 1{,}000$ lbs., above the knee brace the shear is the horizontal component of the stress in 10–X = 10′–X = 9,000 lbs. The maximum bending moment in the post is at the foot of the leeward knee brace and is $M = 1{,}000 \times 13\frac{1}{3} = 13{,}333$ ft.-lbs. For further explanation see the author's "The Design of Steel Mill Buildings."

(*c*) **Results.**—The stresses in the members do not follow the usual rules for trusses loaded with vertical loads; the top chord is partly in tension and partly in compression, while the bottom chord is in compression. The bent should be designed for the wind load stresses combined with the dead load and the minimum snow load stresses, for the wind load and the dead load stresses, or for the wind load and the dead load stresses, whichever combination produces maximum stresses or reversals of stresses.

The stresses in the posts are calculated by dropping the points 1, 2, 10 and 11 to the points 1′, 2′, 10′ and 11′, respectively, on the load line, or on load line produced. The stresses in the windward post are 1′–Y and 2′–3, while the stresses in the leeward post are 11′–Y and 9–10′. The maximum shear in the leeward post is above the knee brace and is 10′–X = 9,000 lbs.



PROBLEM 24A. WIND LOAD STRESSES IN A TRANSVERSE BENT BY GRAPHIC RESOLUTION.

(*a*) **Problem.**—Given a transverse bent, span 40′ 0″, pitch of roof ¼, height of posts 20′ 0″, posts pin-connected at the base, wind load 20 lbs. per square foot normal to the sides and the normal component of a horizontal wind load of 30 lbs. per square foot on the roof. (The normal load on the roof for a horizontal wind load of 30 lbs., is 22½ lbs. per sq. ft., see "Steel Mill Buildings.") Calculate the wind load stresses in the transverse bent by graphic resolution. Scale of bent, 1″ = 10′ 0″. Scale of loads, 1″ = 3,000 lbs.

PART II.

THE DESIGN OF HIGHWAY BRIDGES.

Introduction.—Highway bridges are built (1) of steel or iron; (2) of steel or iron and timber; (3) of timber; (4) of masonry or concrete; and (5) of reinforced concrete.

Steel Bridges.—Steel highway bridges may for convenience be divided into (a) short span bridges, including beam, leg and low truss bridges; (b) high truss bridges; and (c) plate girders. Truss bridges are made with pin-connected joints—"pin-connected," or with riveted joints—"riveted."

Combination Bridges.—Combination bridges have timber upper chords, posts and struts, and steel or iron tension members. Combination bridges are commonly made with the Pratt type of truss, and may have parallel or inclined chords. Combination bridges are used only where timber is cheap, and steel and iron are relatively expensive.

Timber Bridges.—Timber is used for trestles, culverts, and occasionally for truss bridges. The Howe truss is usually made with timber upper and lower chords and diagonal struts, the vertical ties being steel or iron rods. Timber bridges are now used only for temporary structures and for locations where there is not sufficient money for good bridges.

Masonry Bridges.—Arch bridges were formerly made of stone masonry or plain concrete and later of reinforced concrete. Masonry bridges, when properly designed and constructed, are permanent structures.

Reinforced Concrete Bridges.—Reinforced concrete is now quite generally used for arch and beam highway bridges, trestles and culverts. Reinforced concrete structures can be built in locations where

it would not be possible to build ordinary masonry arches, and can usually be built for less money. Reinforced concrete structures require great care, and when so constructed are permanent.

The design of different types of highway bridges will be discussed in detail in Part II.

CHAPTER X.

SHORT SPAN STEEL HIGHWAY BRIDGES.

Introduction.—The term short span highway bridges will be assumed to include beam, leg and low truss bridges.

BEAM BRIDGES.—Beam bridges are made by placing steel beams side by side with the ends resting on the abutments. The roadway floor is usually made of planks laid transversely on the tops of the beams. The spacing of the beams depends upon the load to be carried and upon the thickness of the floor planks, and varies from 2 to 3 feet. A common rule for the thickness of oak floor planks is that the plank shall have at least one inch in thickness for each foot of spacing of the joists or stringers. The outside beams carry a smaller load than the intermediate beams and are usually steel channels. The intermediate beams are steel I beams. It is commonly specified that rolled beams shall have a depth not less than $1/30$ the span.

A steel beam bridge, as designed by the Pittsburg Bridge Company, is shown in Fig. 140. This bridge was designed for a roadway 15 feet wide. The hub guard or fence railing is composed of two angles laced, while the posts are single angles braced as shown. The bottom flanges of the beams are braced by means of a 4-in. channel as shown. The lower flanges of the beams are not punched but are fastened to the channel braces by means of lugs. The floor planks are spiked to the $3'' \times 8''$ spiking strips, which are in turn bolted to the upper flanges of the beams. The shims that are placed between the felloe guard and the floor planks are of doubtful utility. This bridge has no lateral bracing, the floor being sufficiently rigid to transfer the wind loads to the ends of the bridge.

Standard steel beam bridges, as designed by the American Bridge Company, are shown in Fig. 141 and Fig. 142. The details of both bridges are the same with the exception of the fence. Angle cross-

braces are used on both bridges in the place of the channel braces shown in Fig. 140. The beams rest directly on the bridge seat of the abutment and not on wall channels as in Fig. 140, although a channel

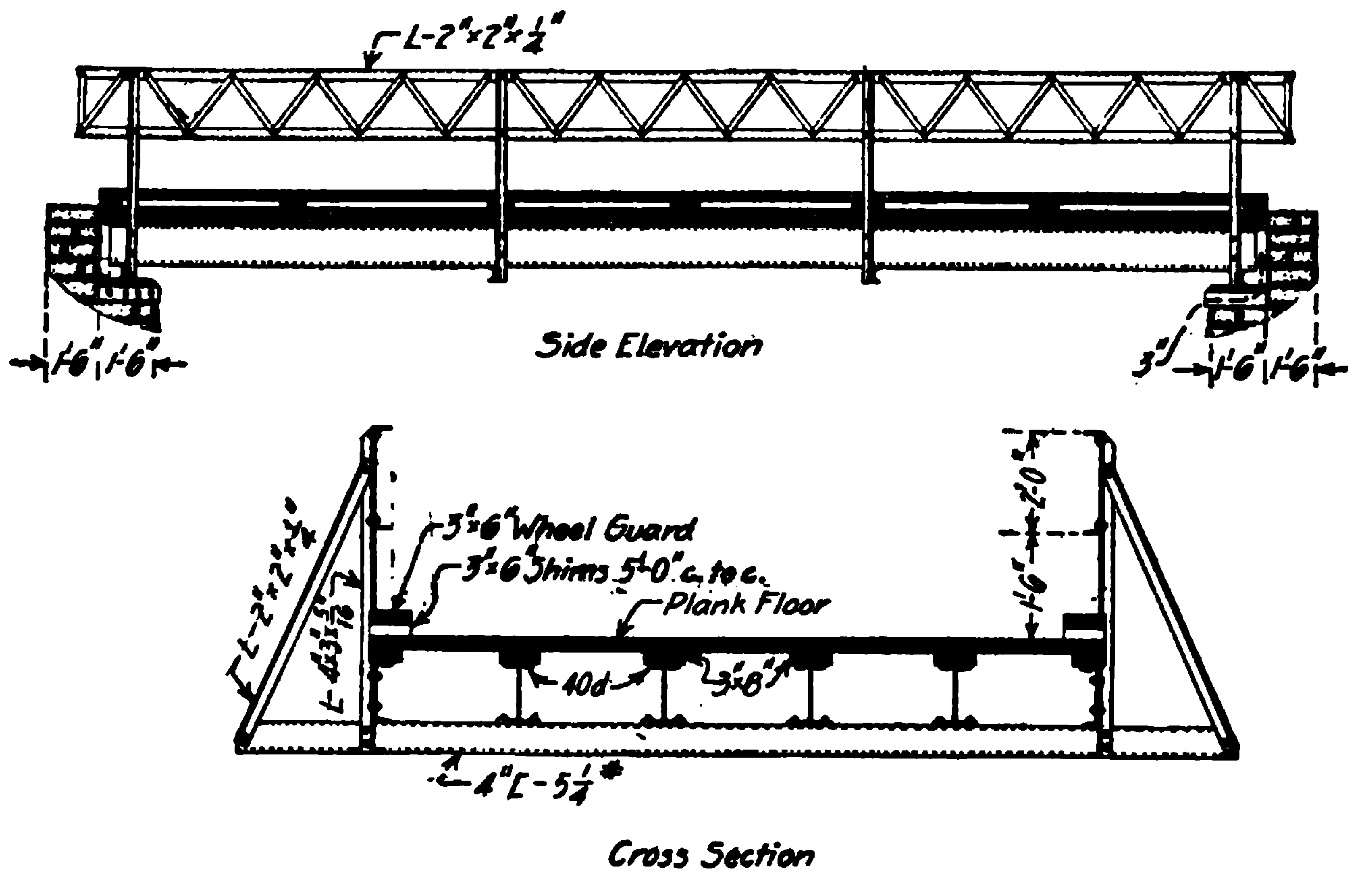


FIG. 140. BEAM BRIDGE. (PITTSBURG BRIDGE COMPANY.)

is sometimes laid on the bridge seat with the legs turned down to carry the beams. The gas pipe rail in Fig. 141 is much cheaper than the lattice rail in Fig. 142. The sizes and spacing of the beams for the standard beam bridges shown in Figs. 141 and 142 for different roadways and spans are given in Table XII (also see Tables VII and VII, Chapter II). The bridges in Table XII were designed for a live load of 125 lbs. per square foot of floor, or a 15-ton road roller. The weight of the railing shown in Fig. 142 may be taken at 33 lbs. per lineal foot of bridge, plus 100 lbs. for each railing post.

In the place of the spiking strips on the tops of the beams, as shown in Figs. 140 to 142, inclusive, spiking strips are sometimes bolted on the sides of the channels and the center I beam, or two channels are used for the center beam with the spiking strip bolted between them. The floor planks are spiked to these spiking strips, and are fastened to the other beams by clinching spikes, which have been driven through the planks, around the top flanges of the beams.

The maximum span for beam bridges is usually given as 40 feet. A better limit for beam spans is 32 feet. Riveted truss bridges should

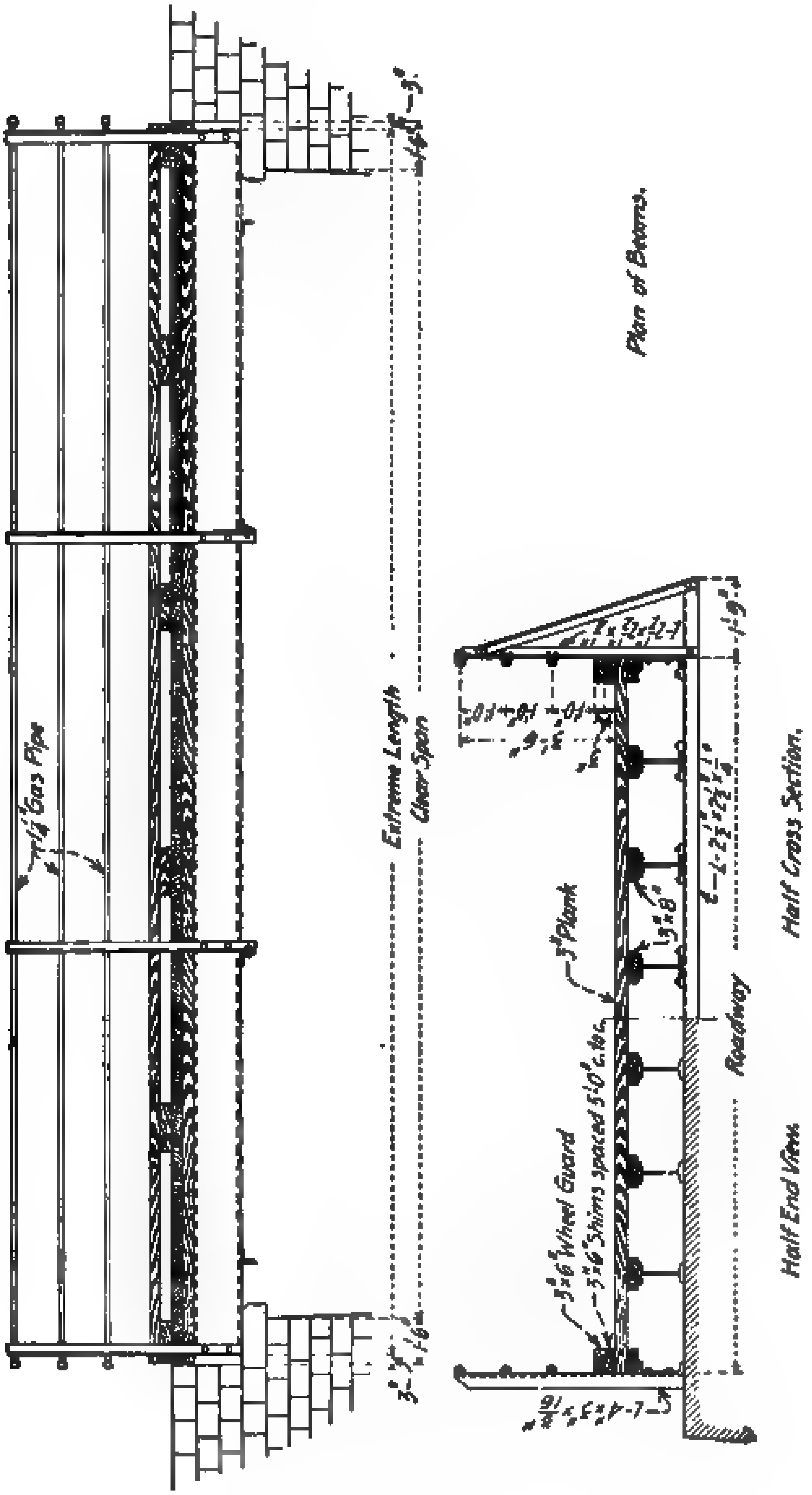


FIG. 141. BEAM BRIDGE. (AMERICAN BRIDGE COMPANY.)

be used for spans of 32 feet and upwards for country bridges, and plate girders for heavy city bridges. Riveted bridges for spans of, say,

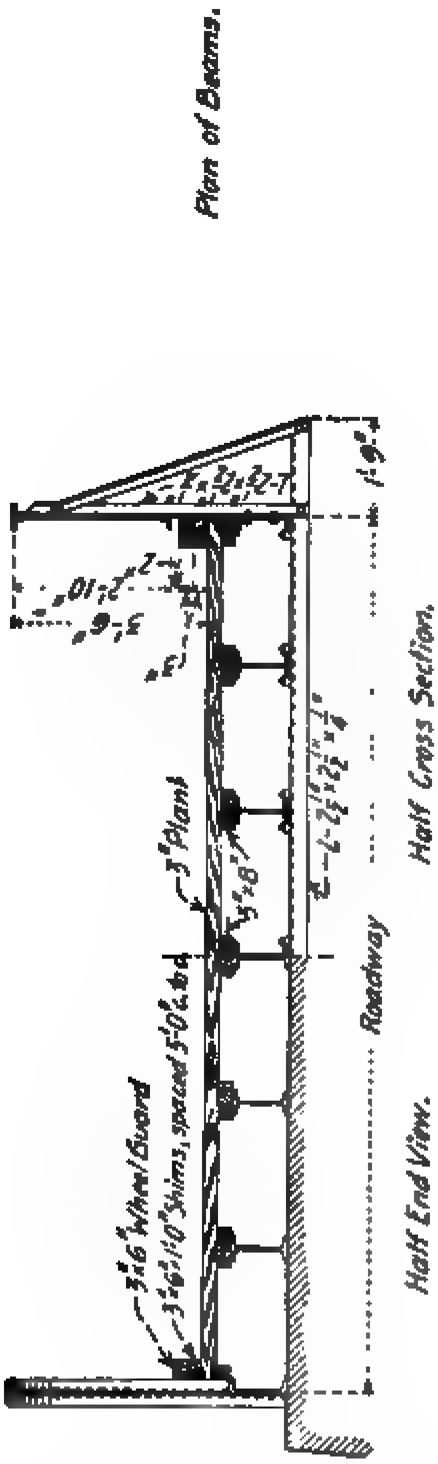


FIG. 142. BEAM BRIDGE (AMERICAN BRIDGE COMPANY.)

40 feet are more economical than beam bridges and will give fully as great a length of service. The ends of beam bridges should always be supported on masonry abutments (see Chapter XV).

A beam bridge with a solid floor is shown in Fig. 143. The bridge consists of six 20" @ 60-lb. I beams, with 4" @ 7½-lb. I beams laid transversely and spaced 2' 2" apart. The concrete floor is reinforced

TABLE XII.

STANDARD BEAM HIGHWAY BRIDGES. (AMERICAN BRIDGE COMPANY, CLASS D LOADING.) ALSO SEE TABLES VII AND VIII, CHAPTER II.

Length out to out of Beams	12' Wide	14' Wide	16' & 18' Wide	20' & 22' Wide
10' to 14'	3-7" I's 15 lb. 2-7" ['s 9¾ lb.	4-7" I's 15 lb. 2-7" ['s 9¾ lb.	5-7" I's 15 lb. 2-7" ['s 9¾ lb.	6-7" I's 15 lb. 2-7" ['s 9¾ lb.
15' to 18'	3-8" I's 18 lb. 2-8" ['s 11¼ lb.	4-8" I's 18 lb. 2-8" ['s 11¼ lb.	5-8" I's 18 lb. 2-8" ['s 11¼ lb.	6-8" I's 18 lb. 2-8" ['s 11¼ lb.
19' to 22'	3-9" I's 21 lb. 2-9" ['s 13¼ lb.	4-9" I's 21 lb. 2-9" ['s 13¼ lb.	5-9" I's 21 lb. 2-9" ['s 13¼ lb.	6-9" I's 21 lb. 2-9" ['s 13¼ lb.
23' to 26'	3-10" I's 25 lb. 2-10" ['s 15 lb.	4-10" I's 25 lb. 2-10" ['s 15 lb.	5-10" I's 25 lb. 2-10" ['s 15 lb.	6-10" I's 25 lb. 2-10" ['s 15 lb.
27' to 30'	3-12" I's 31½ lb. 2-12" ['s 20½ lb.	4-12" I's 31½ lb. 2-12" ['s 20½ lb.	5-12" I's 31½ lb. 2-12" ['s 20½ lb.	6-12" I's 31½ lb. 2-12" ['s 20½ lb.
31' to 35'	3-15" I's 42 lb. 2-15" ['s 33 lb.	4-15" I's 42 lb. 2-15" ['s 33 lb.	5-15" I's 42 lb. 2-15" ['s 33 lb.	6-15" I's 42 lb. 2-15" ['s 33 lb.
36' to 40'	5-18" I's 55 lb.	6-18" I's 55 lb.	7-18" I's 55 lb.	7-18" I's 55 lb.

Note:- Clear Span under Coping is 2 feet less than length out to out of beams.

near the bottom with expanded metal. A beam bridge of three spans is shown in Fig. 144. The intermediate supports are steel bents braced transversely. This bridge with its abutments makes a very satisfactory structure.

LEG BRIDGES.—Beam and truss bridges are sometimes supported on steel legs in the place of abutments. A steel leg beam bridge is shown in Fig. 145. The legs are composed of I beams supported on a steel channel mudsill. The backing is a $\frac{3}{16}$ inch steel plate. The steel legs are usually bolted to timber mudsills, and have either a plank or a stone slab backing. The legs should be designed to carry the thrust of the filling in addition to the live and dead load on one-half

of the span. (For methods of calculation of thrust due to the earth filling, see the author's "The Design of Walls, Bins and Grain Elevators.") Truss leg bridges should be built with stiff lower chords designed to take the thrust due to the filling. Leg bridges, unless very carefully designed and constructed, are not to be recommended.

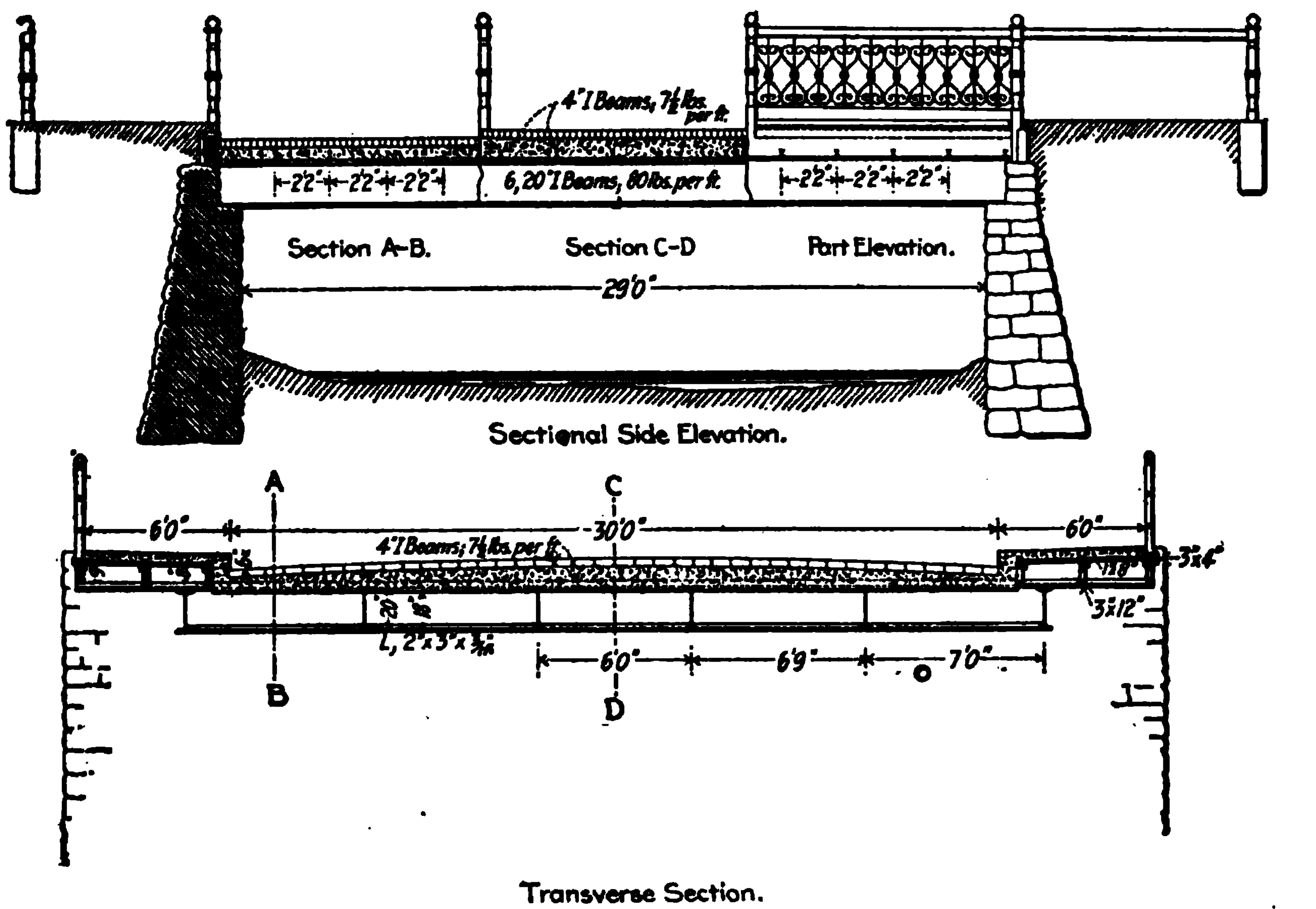


FIG. 143. BEAM BRIDGE WITH SOLID FLOOR.

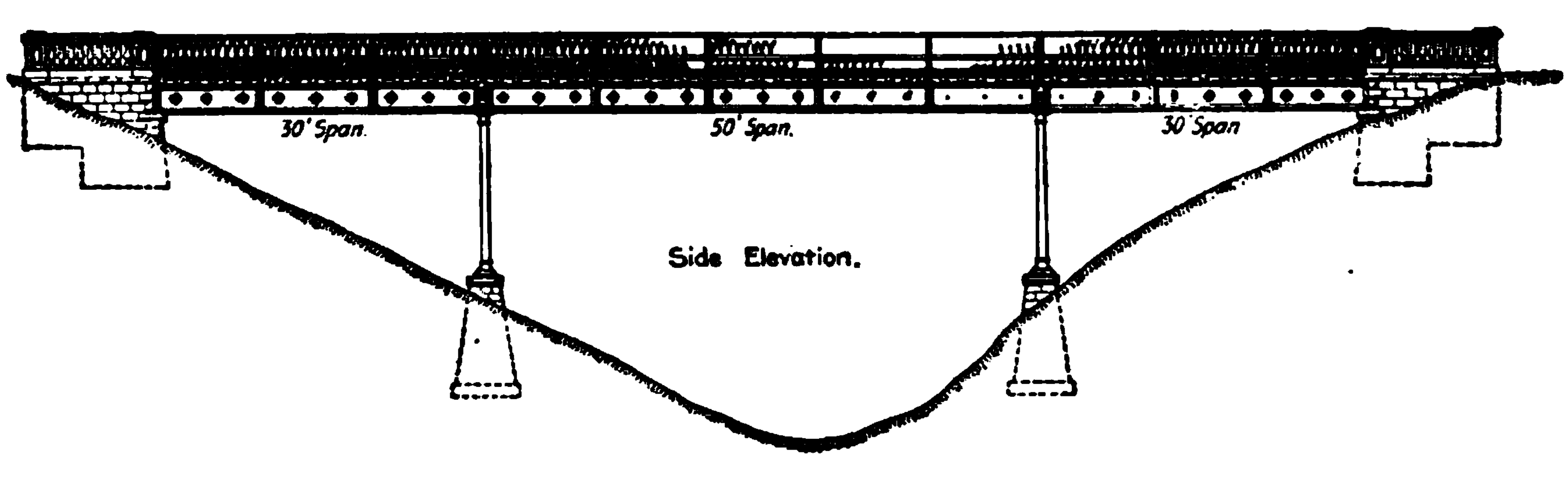


FIG. 144. ORNAMENTAL BEAM BRIDGE WITH INTERMEDIATE POSTS.

LOW TRUSS BRIDGES.—Low truss highway bridges are used for spans of from 30 to 80 feet, and special designs to about 100 feet. The trusses may have either pin-connected or riveted joints. The

trusses may have either half-hip, as in Fig. 146, or full slopes, as in Fig. 147, and may be either of the Warren type, as in Fig. 146, or of the Pratt type, as in Fig. 147. Low truss highway bridges should always be made with riveted connections unless great care is used in the design of pin-connected bridges. The cost is practically the same for the two types. For a discussion of the relative advantages and disadvantages of riveted and pin-connected highway bridges, see Chapter XX.

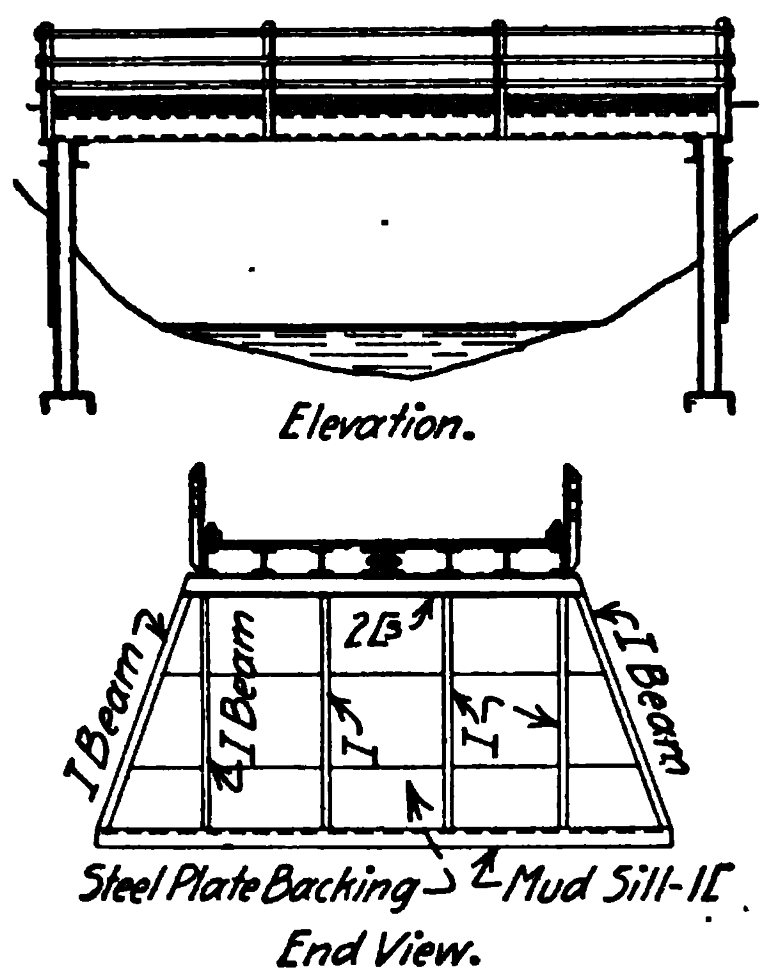


FIG. 145. BEAM LEG BRIDGE WITH STEEL PLATE BACKING.

Riveted Low Truss Bridges.—Low riveted bridges are made with either Warren or Pratt trusses, the Warren truss usually being preferred. The upper chords of short and medium length spans are usually made of two angles, placed back to back as in Figs. 146 and 147, or box-laced, as in Fig. 148. For longer spans the upper chords may be made of two angles and a plate, two channels laced, or two channels with a top cover plate and lacing on the bottom side of the member. The lower chords and the web members are made of two angles placed in the same relative positions as in the upper chords.

A four-panel Warren truss highway bridge, designed by the Gillette-Herzog Mfg. Co., is shown in Fig. 146. The chords and the diagonal

Cross Section.

Fig. 146. Low Warren Riveted Highway Bridge. (Gillette-Herzog Mfg. Co.)

web members are made of two angles placed back to back, forming a T-section, while the posts are made of two angles "starred." The

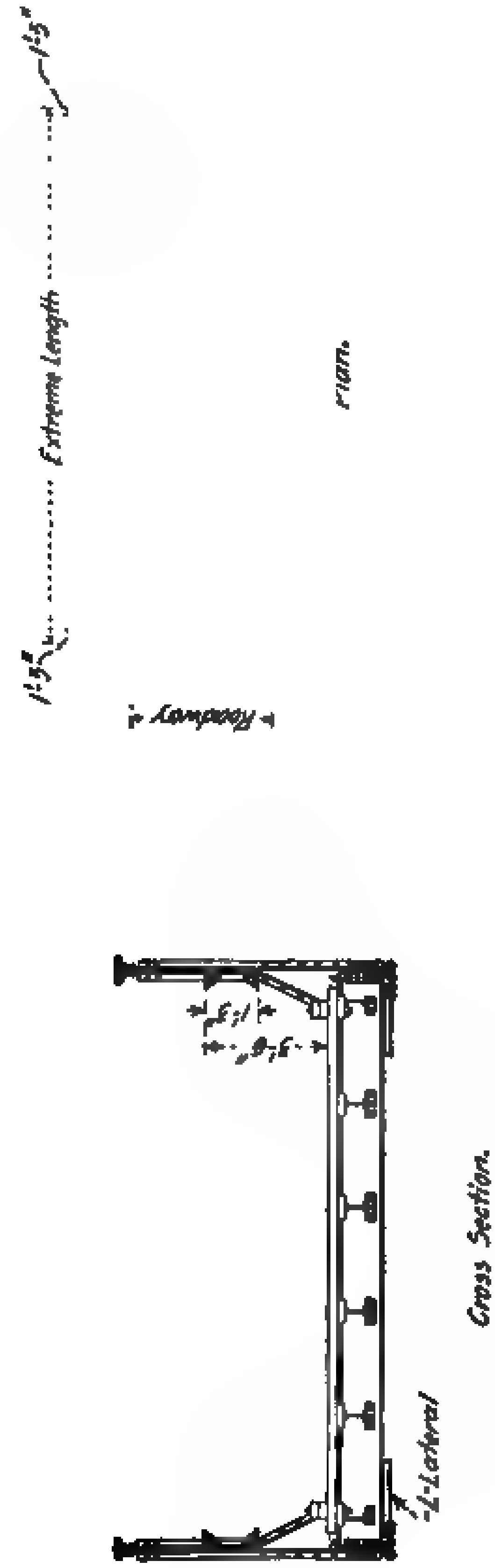


FIG. 147. LOW PRATT RIVETED HIGHWAY BRIDGE. (AMERICAN BRIDGE COMPANY.)

floorbeams are riveted below the lower chord and are rolled **I** beams. The joists are carried directly on the tops of the floorbeams and are composed of four channels and four **I** beams placed as shown. The details of the floor are clearly shown. The lower laterals are made of single angles with riveted connections. The upper chord is braced at the panel points with angle braces.

A four-panel Pratt low truss highway bridge, designed by the American Bridge Company, is shown in Fig. 147. The upper and the lower chords and the posts are made of two angles placed back to back,

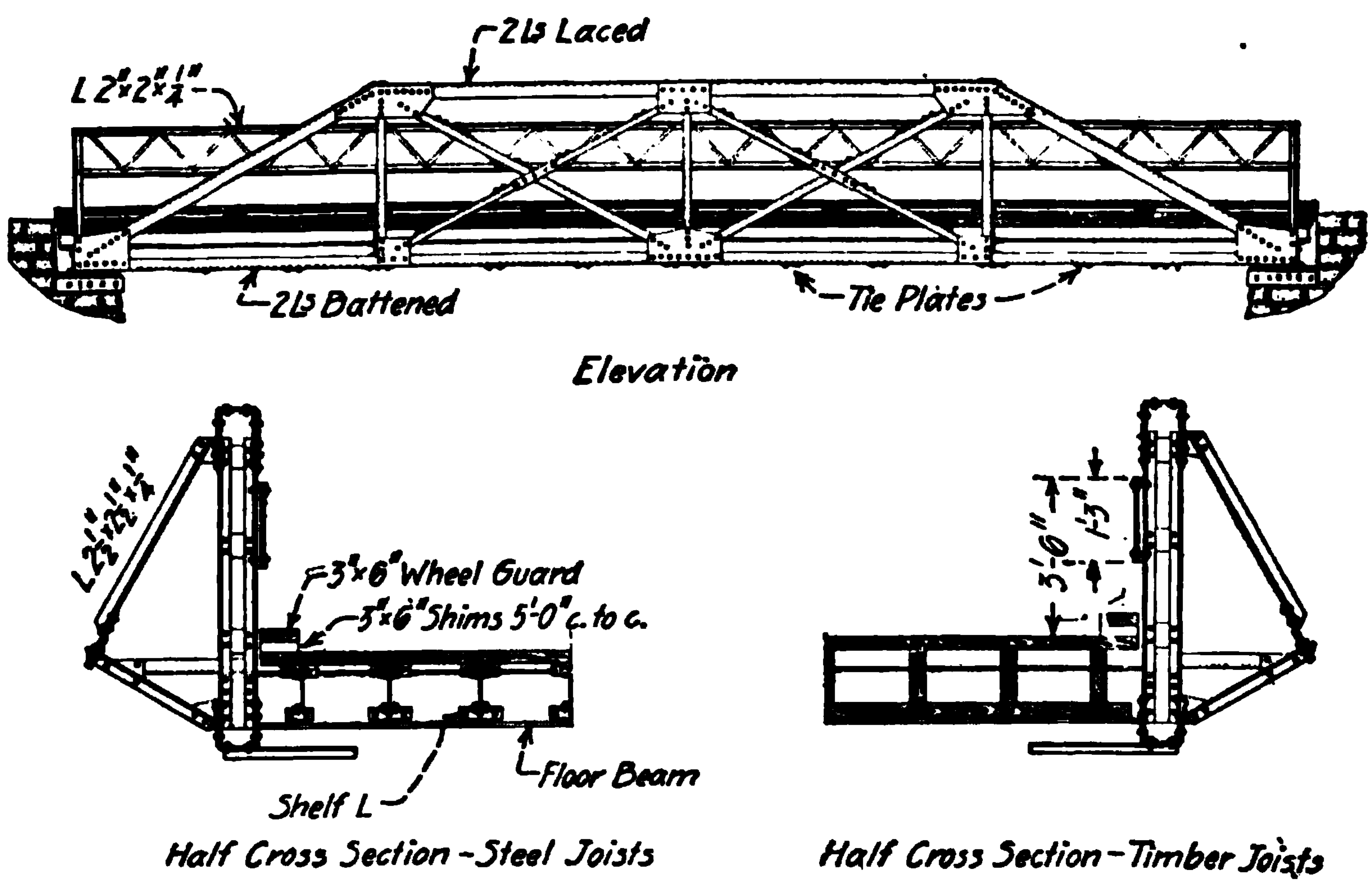


FIG. 148. LOW PRATT RIVETED HIGHWAY BRIDGE. (PITTSBURG BRIDGE COMPANY.)

forming a T-section. The diagonals are single angles acting in tension, only. The trusses are braced by bending one of the angles of the posts. The floorbeams are riveted to the posts above the lower chords, and are rolled **I** beams. The joists are carried on shelf angles riveted to the webs of the floorbeams as shown. The lower lateral systems are made of single angles with riveted connections. The American Bridge Company also builds low truss bridges of other types (see Fig. 156).

A four-panel Pratt low truss highway bridge, designed by the Pittsburg Bridge Co., is shown in Fig. 148. The upper chords are made of

two angles laced, the posts and the lower chords are made of two angles fastened together with tie plates, while the diagonal members are made of single angles. The upper chord is braced at the panel

FIG. 149. DETAILS OF LOW TRUSS RIVETED HIGHWAY BRIDGE. (AMERICAN BRIDGE COMPANY.)

points by an angle brace as shown. The floorbeams are riveted above the lower chords. Steel joists are carried on shelf angles riveted to the rolled floorbeams, while timber joists are carried on a timber bolted to the floorbeams. The lower laterals are single angles.

Details.—Details of a riveted low truss highway bridge with the floorbeams riveted below the lower chords, as designed by the American Bridge Company, are shown in Fig. 149. The end shoe is bolted

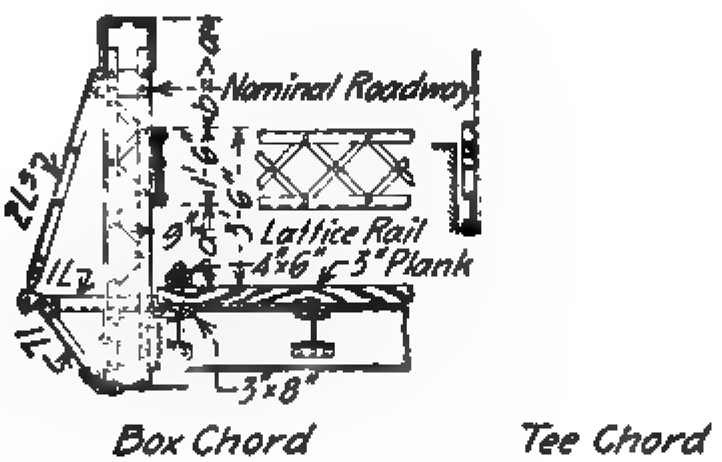


FIG. 150. DETAILS OF LOW TRUSS RIVETED HIGHWAY BRIDGE. (AMERICAN BRIDGE COMPANY.)

to the bridge seat by means of anchor bolts. The holes in the bearing plates of the shoes should be slotted at one end to permit movement due to changes in temperature. Sliding plates should be provided on

floorbeams are riveted below the lower chord and are rolled **I** beams. The joists are carried directly on the tops of the floorbeams and are composed of four channels and four **I** beams placed as shown. The details of the floor are clearly shown. The lower laterals are made of single angles with riveted connections. The upper chord is braced at the panel points with angle braces.

A four-panel Pratt low truss highway bridge, designed by the American Bridge Company, is shown in Fig. 147. The upper and the lower chords and the posts are made of two angles placed back to back,

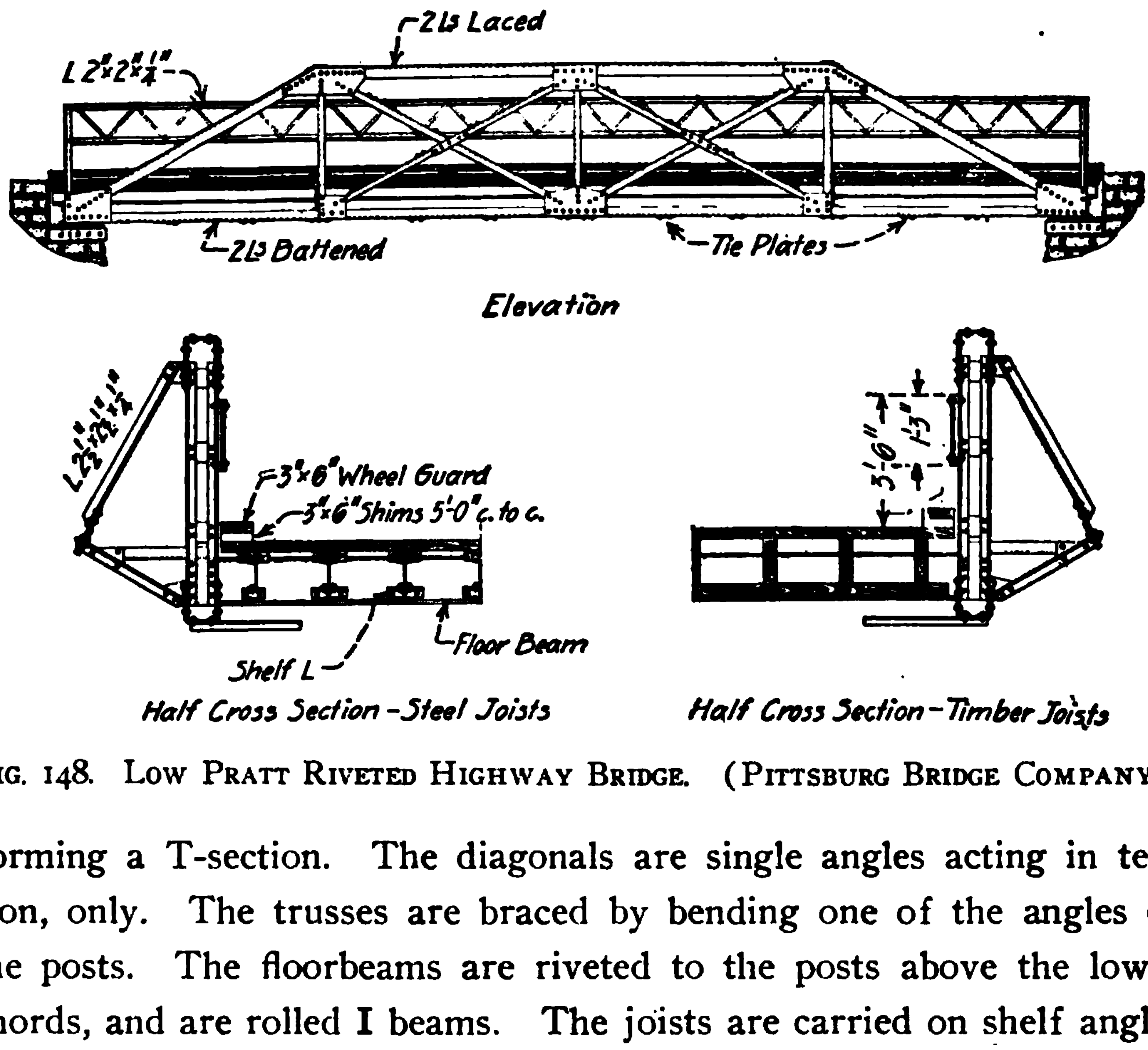


FIG. 148. LOW PRATT RIVETED HIGHWAY BRIDGE. (PITTSBURG BRIDGE COMPANY.)

forming a T-section. The diagonals are single angles acting in tension, only. The trusses are braced by bending one of the angles of the posts. The floorbeams are riveted to the posts above the lower chords, and are rolled **I** beams. The joists are carried on shelf angles riveted to the webs of the floorbeams as shown. The lower lateral systems are made of single angles with riveted connections. The American Bridge Company also builds low truss bridges of other types (see Fig. 156).

A four-panel Pratt low truss highway bridge, designed by the Pittsburg Bridge Co., is shown in Fig. 148. The upper chords are made of

two angles laced, the posts and the lower chords are made of two angles fastened together with tie plates, while the diagonal members are made of single angles. The upper chord is braced at the panel

FIG. 149. DETAILS OF LOW TRUSS RIVETED HIGHWAY BRIDGE. (AMERICAN BRIDGE COMPANY.)

points by an angle brace as shown. The floorbeams are riveted above the lower chords. Steel joists are carried on shelf angles riveted to the rolled floorbeams, while timber joists are carried on a timber bolted to the floorbeams. The lower laterals are single angles.

Details.—Details of a riveted low truss highway bridge with the floorbeams riveted below the lower chords, as designed by the American Bridge Company, are shown in Fig. 149. The end shoe is bolted

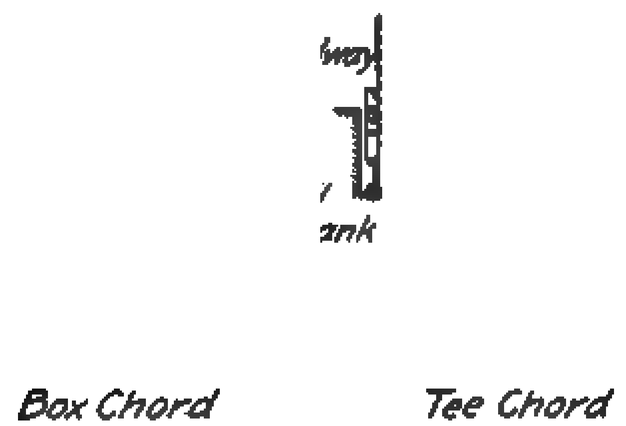


FIG. 150. DETAILS OF LOW TRUSS RIVETED HIGHWAY BRIDGE. (AMERICAN BRIDGE COMPANY.)

to the bridge seat by means of anchor bolts. The holes in the bearing plates of the shoes should be slotted at one end to permit movement due to changes in temperature. Sliding plates should be provided on

the expansion end; the surfaces of the bearing and sliding plates in contact being planed.

Details of riveted low truss highway bridges with box- and with tee-chords, and with floorbeams riveted below the lower chords, as

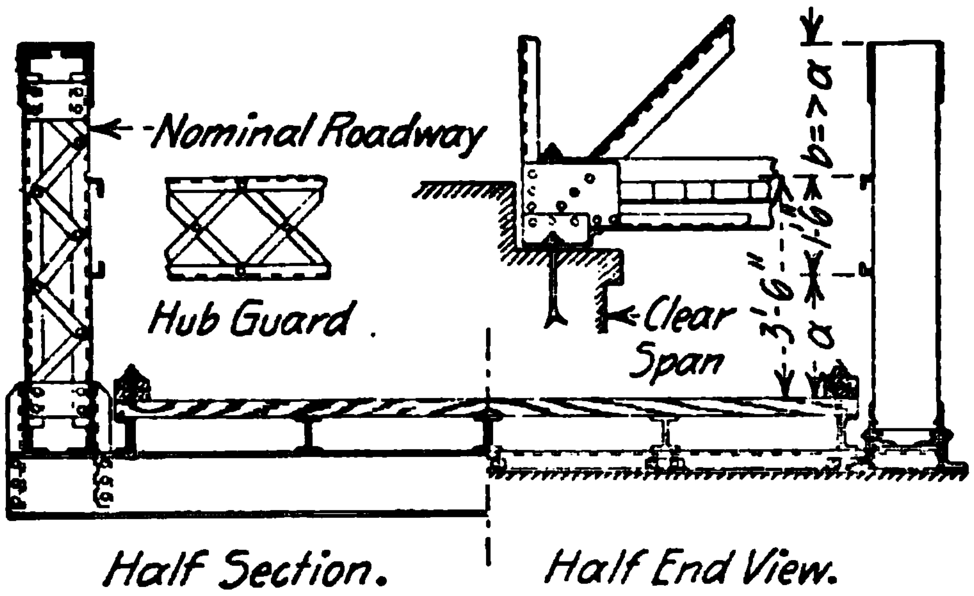


FIG. 151. DETAILS OF LOW TRUSS RIVETED HIGHWAY BRIDGE. (AMERICAN BRIDGE COMPANY.)

designed by the American Bridge Company, are shown in Fig. 150. Details of a riveted low truss highway bridge with box-chords and with suspended floorbeams are shown in Fig. 151. It will be noted

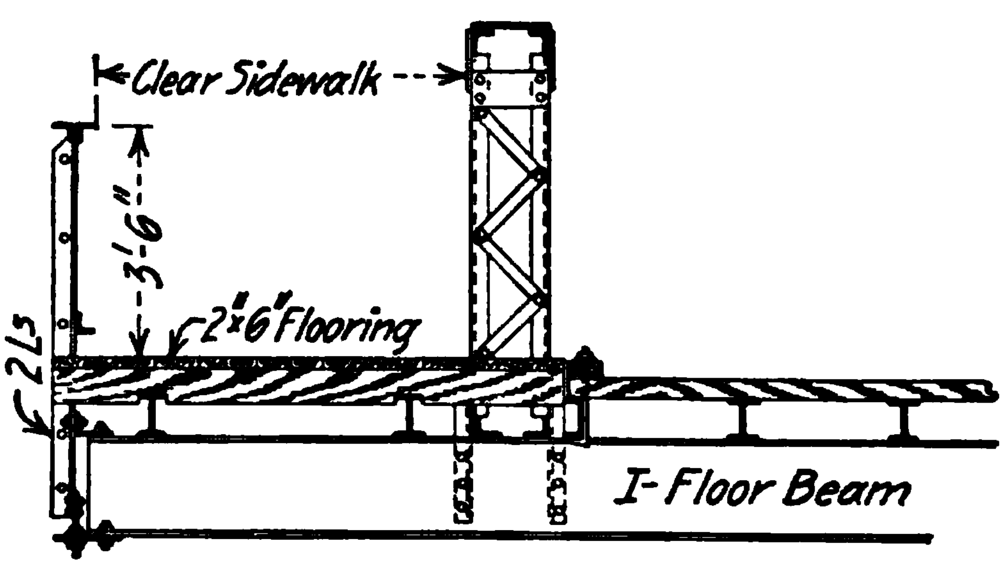


FIG. 152. DETAILS OF LOW TRUSS RIVETED HIGHWAY BBIDGE WITH SIDEWALK. (AMERICAN BRIDGE COMPANY.)

that no side braces are provided in this design. The same method of suspending the floorbeams is shown in Fig. 152 for a low truss bridge with sidewalks. Where side braces are not used the posts should be made wider than where braces are used.

A riveted low truss highway bridge, as designed for Portage County, Ohio, is shown in Figs. 153 and 154. The chords are made of two

channels laced. The floorbeams are built-up plate girders and are suspended below the lower chords. The joists and the floor are made of

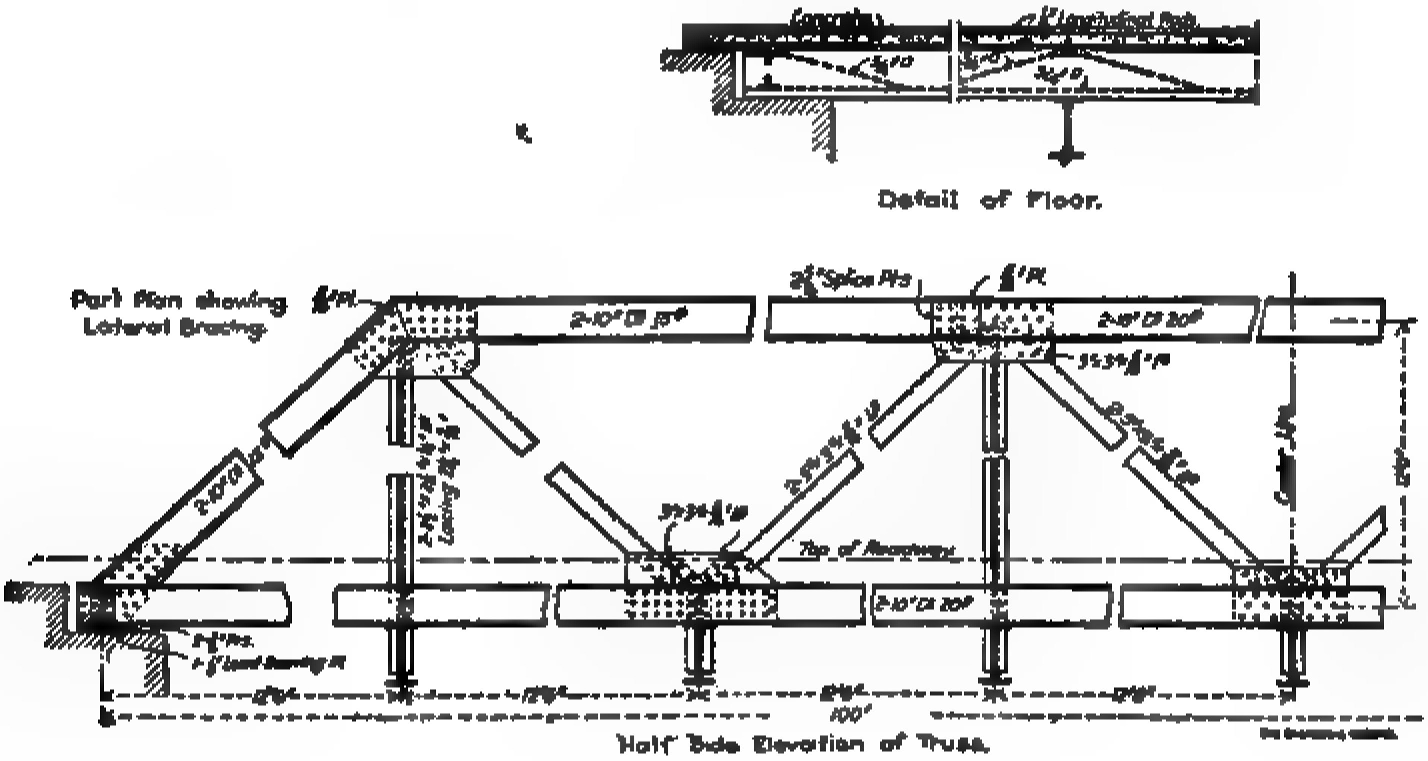


FIG. 153. ELEVATION OF RIVETED HIGHWAY BRIDGE, PORTAGE COUNTY, OHIO.

Half Transverse Sectional Elevation

FIG. 154. SECTION OF RIVETED HIGHWAY BRIDGE, PORTAGE COUNTY, OHIO.

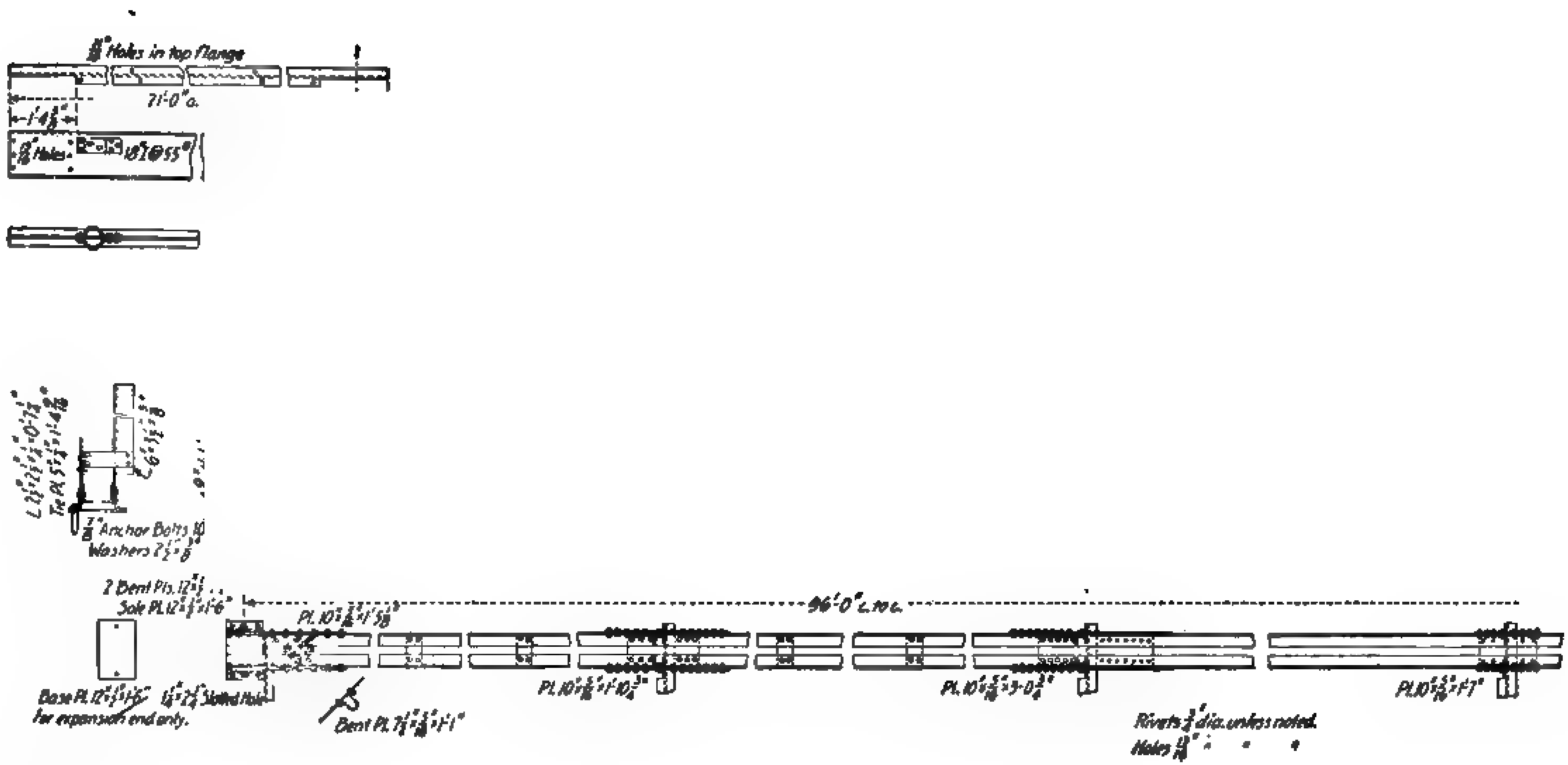
reinforced concrete. This bridge was designed by the Osborn Engineering Company, and is described in Engineering Record, April 25, 1903.

The riveted low truss highway bridge with an inclined upper chord shown in Figs. 155 to 157, is built by the American Bridge Com-

FIG. 155. LOW TRUSS RIVETED HIGHWAY BRIDGE WITH INCLINED CHORDS. (AMERICAN BRIDGE CO.)

FIG. 156. LOW TRUSS RIVETED HIGHWAY BRIDGE WITH INCLINED CHORDS. END VIEW. (AMERICAN BRIDGE CO.)

pany for locations requiring an artistic and serviceable bridge at a moderate cost. This bridge has been built with six panels and with



(207) FIG. 157. DETAILS OF LOW TRUSS RIVETED HIGHWAY BRIDGE WITH INCLINED CHORDS. (AMERICAN BRIDGE COMPANY.)

spans of 90, 96 and 102 feet. The bridge in Fig. 157 has a 20-ft. roadway and was designed for a dead load of 930 lbs. per lineal foot of bridge, and a live load of 2,400 lbs. per lineal foot of bridge. The total weight of the steel in this bridge, exclusive of joists and fence is, approximately, 57,000 lbs. The floorbeams are rolled I beams and are riveted below the chords. The top chords are made of two channels with a top cover plate, the lower edges of the channels being fastened together with tie plates—lacing is much better practice. The bottom chord is composed of two angles, with tie plates—tie plates are all right for this member. The web members are made of 2 or 4 angles laced, as shown. Rods, not shown, are used for the lower lateral system.

Pin-connected Low Truss Bridges.—Pin-connected low truss highway bridges are commonly built of the Pratt type, with either half-hip or full slope end-posts. The upper chords of pin-connected low truss highway bridges are made of two channels and a top cover plate, or of two channels laced; the posts are usually made of four

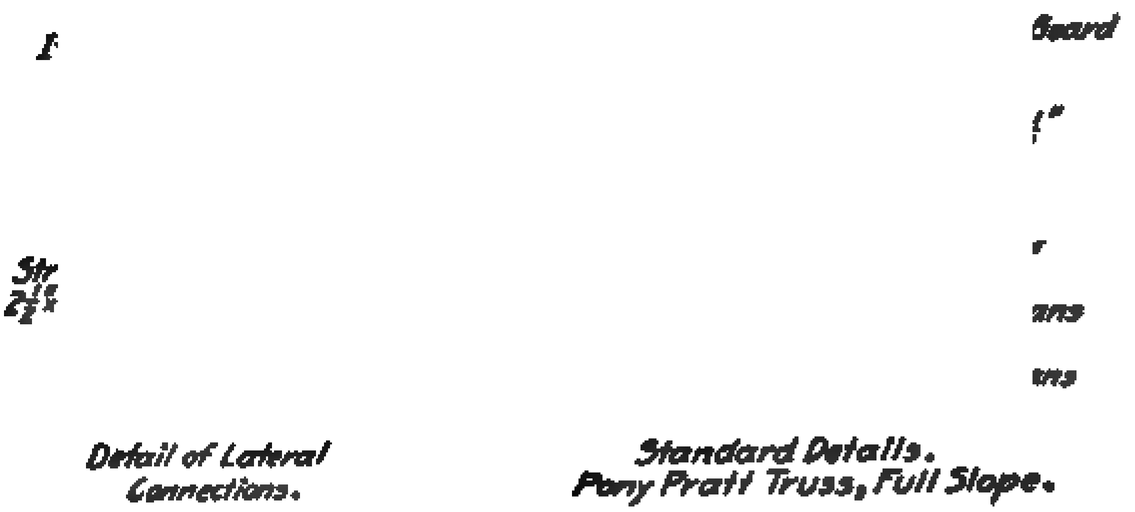


FIG. 158. LOW PRATT FULL SLOPE PIN-CONNECTED HIGHWAY BRIDGE. (AMERICAN BRIDGE COMPANY.)

angles laced or battened; while the tension members are made of rods or eye-bars. The posts and the chords should be made very wide and should be securely fastened to the floorbeams, or side braces should be used. The details of the American Bridge Company's full slope, low truss Pratt highway bridge with the floorbeams riveted

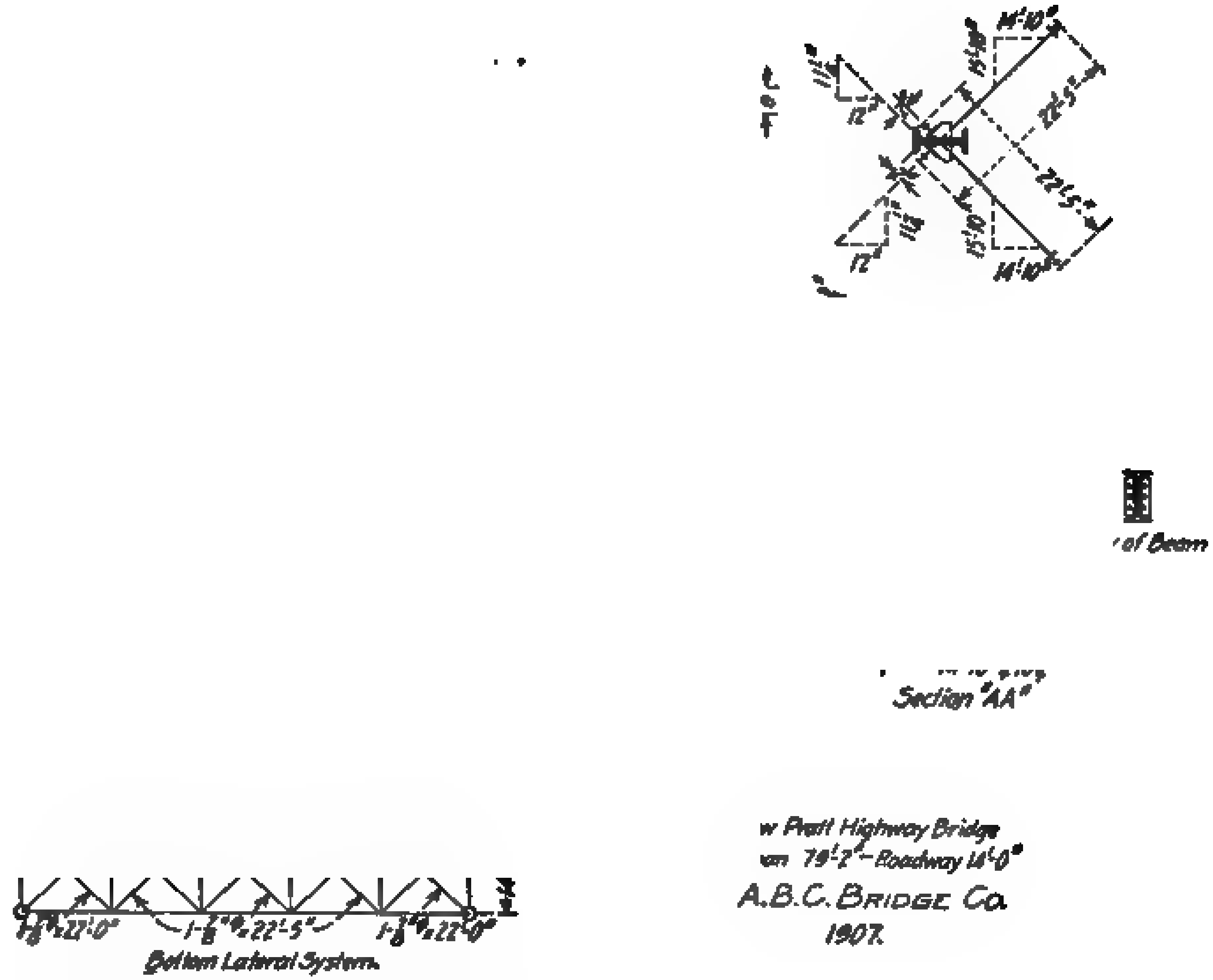


Fig. 159. Low Full Slope "Fish-bellied" Pratt Truss Highway Bridge

below the lower chords are shown in Fig. 158. Rolled beams are used for floorbeams, while the lower laterals are made of rods with screw ends.

The principal objection to pin-connected low truss bridges is that the vertical trusses are usually not sufficiently braced, and lack lateral stability. The "fish-bellied" truss bridge shown in Fig. 159 with the floorbeams riveted above the lower chords, is a decided improvement upon the usual type of low truss bridge. This bridge is very rigid and makes a very satisfactory structure. In Fig. 159 the lower chord pins, beginning with the left end, are called L_0, L_1, L_2, etc., while the upper chord pins are called U_1, U_2, etc., as shown. The top chords and end-posts are made of two channels and a top cover plate, with tie plates on the bottom of the member. The posts are made of four angles; eye-bars are used for the lower chords and main diagonals, while rods are used for the main ties and the diagonals in the middle panels. The joists are carried directly on the tops of the floorbeams.

Temperature Changes.—The expansion ends of low truss bridges should be placed on sliding plates. Rollers are not commonly used for low truss highway bridges.

Weight of Low Truss Bridges.—The weights of low truss highway bridges are given in Chapter II.

Length of Span.—The American Bridge Company's standards include the following lengths of span for the different types of low truss bridges:

TABLE XIII.

LOW TRUSS SPANS USED BY AMERICAN BRIDGE COMPANY.

Type of Truss.	Span in Feet.
Low Warren riveted truss with parallel chords, box-section....	30 to 85
Low Warren riveted trusses with inclined chords..............	90 to 102
Low Warren riveted trusses with parallel chords, T-section......	36 to 75
Low Pratt riveted truss, box-chord..........................	30 to 90
Low Pratt riveted truss, T-section..........................	36 to 60
Low Pratt full-slope pin-connected trusses....................	36 to 90
Low half-hip pin-connected trusses..........................	36 to 56

The Gillette-Herzog Mfg. Company's standards for riveted Warren low truss bridges include spans from 32 to 75 feet. The economical limit for low truss spans is at 75 or 80 feet.

Depth of Truss.—The American Bridge Company's standards for low truss bridges include the following:

TABLE XIV.

DEPTHS OF LOW TRUSSES USED BY AMERICAN BRIDGE COMPANY.

RIVETED TRUSSES.

SPAN, FEET.	NUMBER OF PANELS.	RATIO OF DEPTH TO PANEL LENGTH.
30 to 33	2	0.30
36 to 45	3	0.35
48 to 60	4	0.40
65 to 85	5	0.50
90 to 102	6	0.30, 0.525, and 0.60

PIN-CONNECTED PRATT TRUSSES.

SPAN, FEET.	NUMBER OF PANELS.	RATIO OF DEPTH TO PANEL LENGTH.	DEPTH. FEET.
36	3	0.50	6.0
39	3	0.46	6.0
42	3	0.4285	6.0
45	3	0.40	6.0
48	4	0.50	6.0
52	4	0.4615	6.0
56	4	0.50	7.0
60	4	0.50	7.5
64	4	0.50	8.0
65	5	0.60	8.8
70	5	0.60	8.4
75	5	0.60	9.0
80	5	0.60	8.5
84	5	0.70	9.8
90	6	0.70	10.5

CHAPTER XI.

High Truss Steel Highway Bridges.

Introduction.—The different types of trusses in use for high truss bridges are described in Chapter I. Through truss bridges with spans of from 80 to 170 feet span are built with parallel chords and with either pin-connected or riveted joints. For spans of from 160 to 220 feet, bridges are usually built of the Pratt type with inclined upper

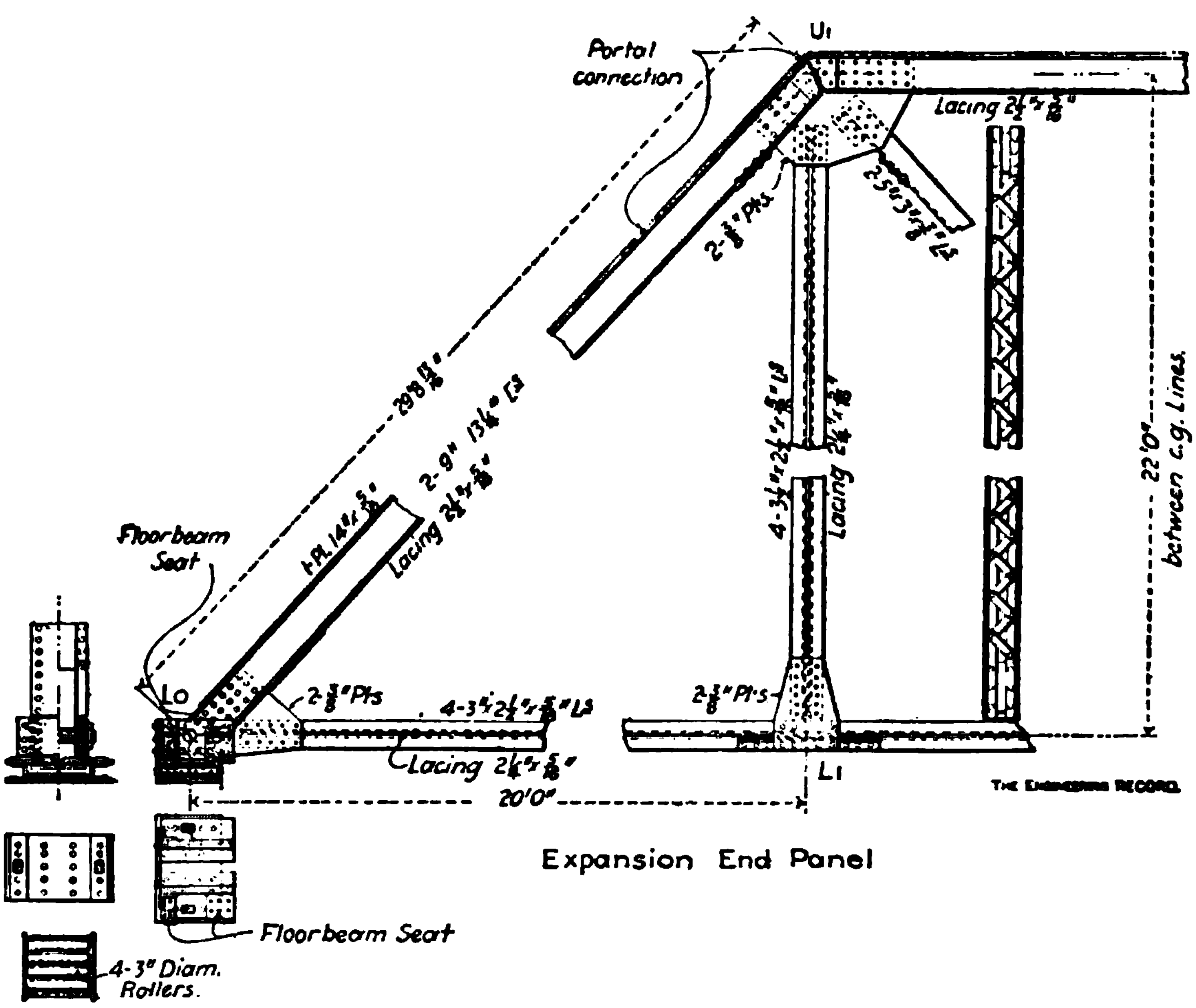
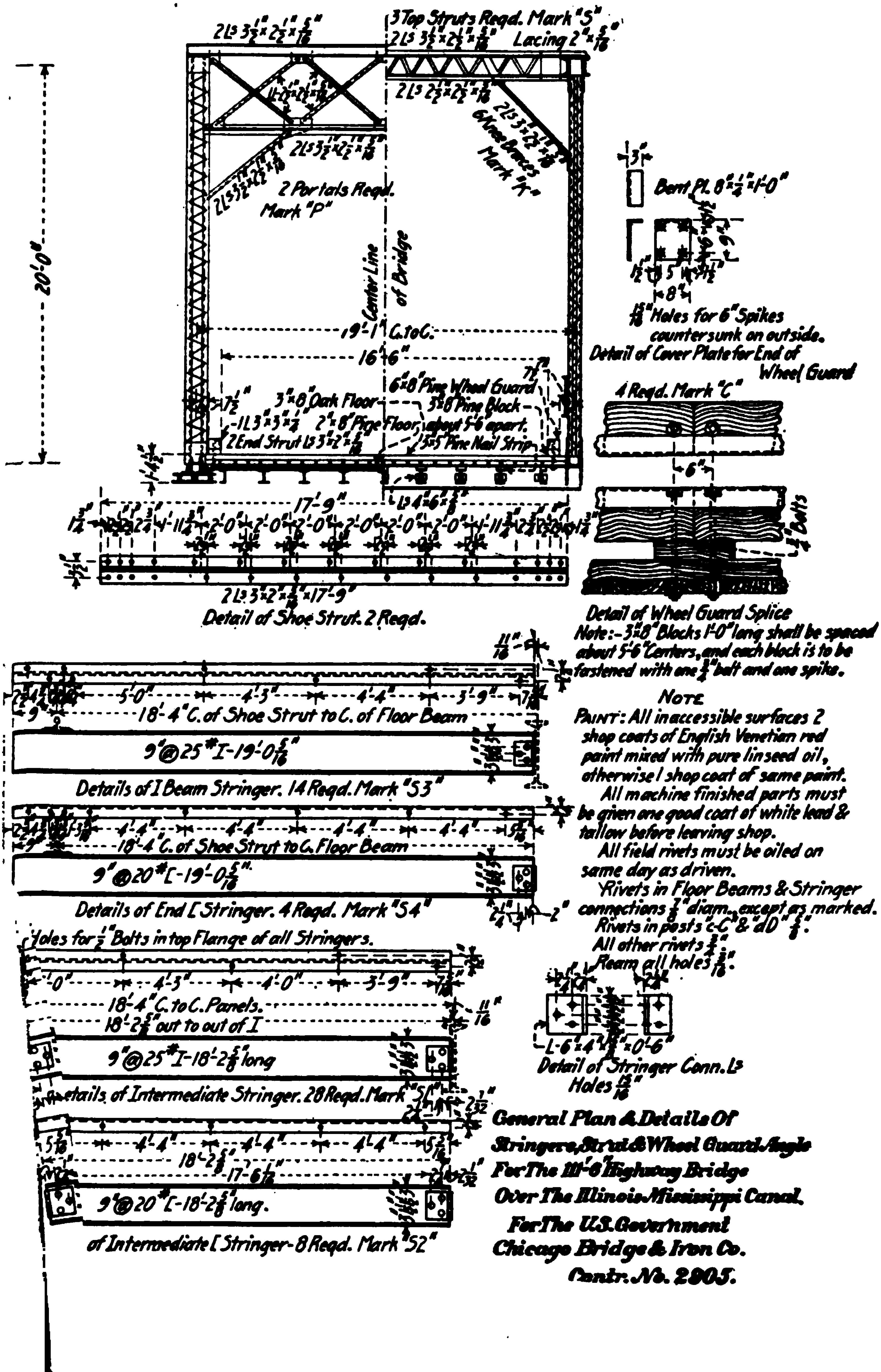


Fig. 160. Through Riveted Pratt Electric Railway Bridge.

chords (Camel-back) trusses. Above 220 feet, bridges are usually built with the Petit type of truss. The above limits are approximate only. High truss pin-connected bridges should never be built with less than five panels.


3 Top Struts Reqd. Mark "S"
Lacing 2"x5/16"
2 Portals Reqd.
Mark "P"
Center Line of Bridge
19'-1" C. to C.
16'-6"
6"x8" Pine Wheel Guard
3"x8" Oak Floor
3"x8" Pine Block
2"x8" Pine Floor, about 5'-6" apart.
2 End Strut
5"x5" Pine Nail Strip
20'-0"
17'-9"
Bent Pl. 8"x1/4"x1'-0"
15/16" Holes for 6" Spikes
countersunk on outside.
Detail of Cover Plate for End of Wheel Guard
4 Reqd. Mark "C"
Detail of Shoe Strut. 2 Reqd.
Detail of Wheel Guard Splice
Note:- 3"x8" Blocks 1'-0" long shall be spaced about 5'-6" Centers, and each block is to be fastened with one 3/4" bolt and one spike.
18'-4" C. of Shoe Strut to C. of Floor Beam
9"@25# I-19'-0 5/16"
Details of I Beam Stringer. 14 Reqd. Mark "S3"
18'-4" C. of Shoe Strut to C. Floor Beam
9"@20# [-19'-0 5/16"
Details of End [Stringer. 4 Reqd. Mark "S4"
NOTE
PAINT: All inaccessible surfaces 2 shop coats of English Venetian red paint mixed with pure linseed oil, otherwise 1 shop coat of same paint.
All machine finished parts must be given one good coat of white lead & tallow before leaving shop.
All field rivets must be oiled on same day as driven.
Rivets in Floor Beams & Stringer connections 7/8" diam., except as marked.
Rivets in posts "c-C" & "dD" 3/4".
All other rivets 3/4".
Ream all holes 15/16".
Holes for 1/2" Bolts in top Flange of all Stringers.
18'-4" C. to C. Panels.
18'-2 5/8" out to out of I
9"@25# I-18'-2 5/8" long
Details of Intermediate Stringer. 28 Reqd. Mark "S1"
Detail of Stringer Conn. Ls
Holes 15/16"
9"@20# [-18'-2 5/8" long.
of Intermediate [Stringer- 8 Reqd. Mark "S2"
General Plan & Details Of
Stringers, Strut & Wheel Guard Angle
For The 111'-6" Highway Bridge
Over The Illinois-Mississippi Canal.
For The U.S. Government
Chicago Bridge & Iron Co.
Contr. No. 2805.

Types of bridges adopted in the American Bridge Company's standards are as follows:

Pratt, pin-connected trusses,	80 to 168-ft. span.
Pratt, riveted trusses,	80 to 168-ft. span.
Warren, quadrangular, riveted trusses,	80 to 152-ft. span.
Inclined chord Pratt (Camel-back), pin-connected trusses,	168 to 220-ft span.
Petit trusses, pin-connected,	220-ft. span and over.

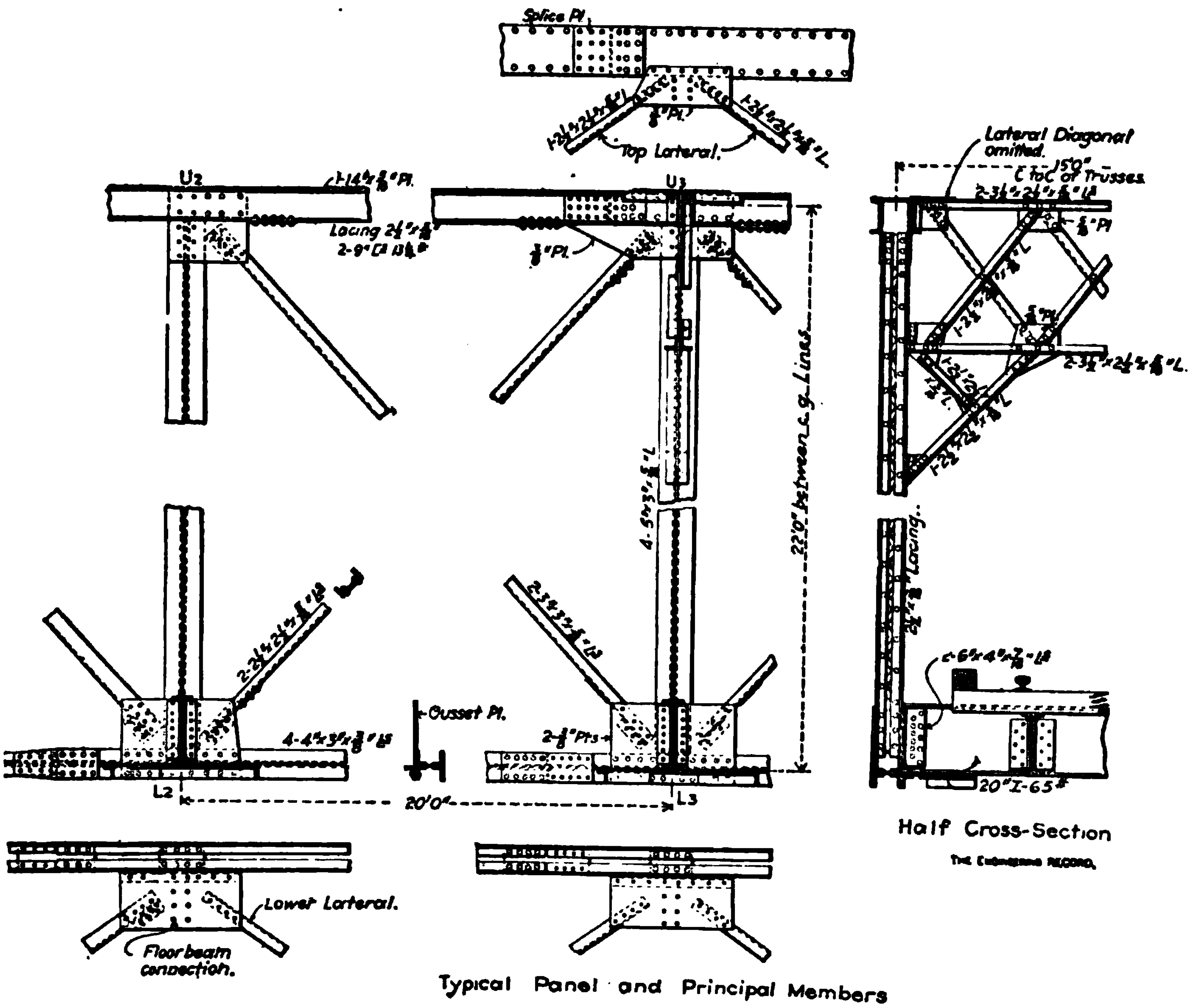


FIG. 161. THROUGH RIVETED PRATT ELECTRIC RAILWAY BRIDGE.

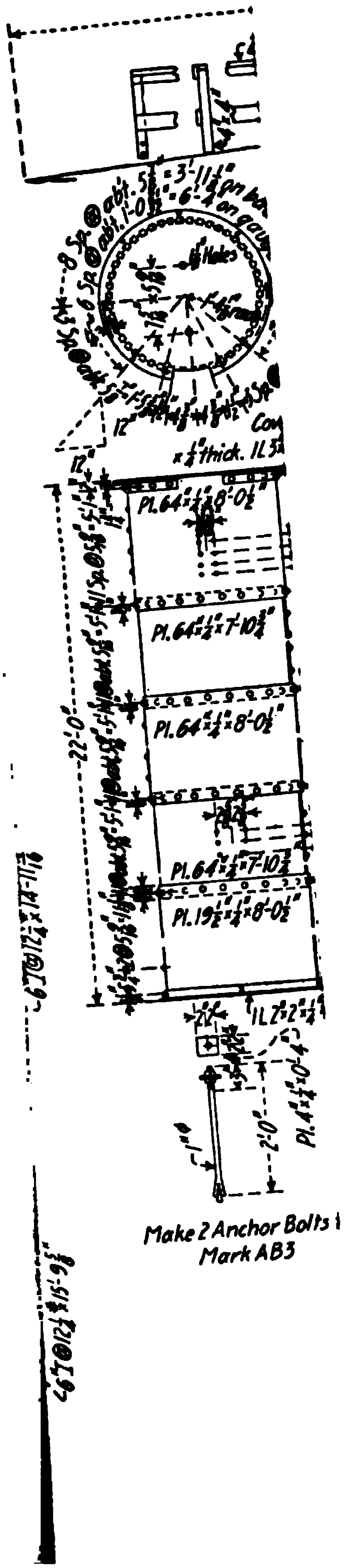
The Gillette-Herzog Mfg. Company's standards included Pratt pin-connected trusses from 80 to 170 ft., Camel-back or Parker trusses, from about 170 to about 200 ft., above this span Petit trusses were used. For a discussion of the relative advantages and disadvantages of riveted and pin-connected highway bridges, see Chapter XX.

RIVETED BRIDGES.—A 120-ft. span riveted Pratt electric railway bridge is shown in Fig. 160 and Fig. 161. The end-posts and upper chords are made of two channels and a top cover plate, the vertical posts and the lower chords are made of four angles, laced, while the diagonals are made of angles. It will be seen that all angles are fastened by both legs. The expansion end of the bridge is carried on two nests of rollers. The floorbeams are 20″ @ 65 lb. I beams, while

FIG. 164. A RIVETED PRATT THROUGH HIGHWAY BRIDGE WITH SIDEWALKS, OVER ILLINOIS AND MISSISSIPPI CANAL.

the stringers are 15″ @ 42 lb. I beams with riveted connections as shown.

The detail shop plans of a 111-ft. 6-in. span riveted Pratt truss highway bridge, as built for the U. S. Government by the Chicago Bridge and Iron Co., Chicago, Ill., are shown in Fig. 162 and Fig. 163. The top chords, the end-posts and the intermediate posts are made of two channels laced on both sides, while the bottom chords, hip verticals and diagonal ties are made of two angles fastened together with tie plates. The floorbeams are 18″ @ 55 lb. I beams and are riveted below the lower chords. The joists are carried by connection angles riveted


Make 2 Anchor Bolts
Mark AB3

to the webs of the floorbeams. The portals and the sway struts are made of angles. The top and bottom laterals are made of adjustable rods. The expansion end of the bridge is carried on two nests of expansion rollers, each nest being composed of four $3\frac{5}{8}''$ rollers. The floor covering is composed of a bottom layer of $2'' \times 8''$ pine plank

FIG. 165. END VIEW OF THE RIVETED PRATT THROUGH HIGHWAY BRIDGE SHOWN IN FIG. 164.

laid transversely and spiked to $3'' \times 5''$ spiking strips that are bolted to the tops of the joists, and a top layer of $3'' \times 8''$ oak, laid diagonally. The $6'' \times 8''$ pine felloe (wheel) guard has its edge protected by a $3'' \times 3'' \times \frac{1}{4}''$ angle. The detailed estimate of the weight of this bridge is given in Table I. The per cent of details in this bridge is quite high, due to the fact that the end-posts and the top chords are made of two channels, laced. Views of a riveted Pratt highway bridge, a duplicate of the bridge described above except that it has two sidewalks is shown in Figs. 164 and 165.

The details of a 95-ft. span quadrangular Warren truss highway bridge are given in Fig. 166. The main members are made of two angles, while the minor members are made of single angles. The floorbeams are made of rolled I beams and are riveted below the chords. The joists are carried directly on the tops of the floorbeams. The top and bottom laterals are made of single angles with riveted connections. The top lateral struts are made of two angles placed back to back, while the portals are made of angles. The bridge is carried on four steel tubular piers 30 inches in diameter, with three piles driven in each tube, the piles being sawed off below the water line and the tube filled with Portland cement concrete. Complete shop drawings of the tubular steel piers are shown. Sliding plates were provided for the expansion end of the bridge in the place of roller nests, as is the usual custom for spans of more than 70 to 80 feet. The complete shop and erection drawings are shown, making it possible to study the details.

PIN-CONNECTED HIGHWAY BRIDGES.—The detail shop drawings of a 160-ft. span pin-connected Pratt truss highway bridge are shown in Fig. 285. The weight of this bridge and the efficiencies of the individual members are calculated in Part III.

The details of a 319-ft. span pin-connected Petit truss highway bridge with a 20-ft. roadway and 2-6 ft. sidewalks are shown in Fig. 167. This bridge was designed as a Cooper's Class B, Specifications 1890. The end-posts and the upper chords are made of two 15″ channels and one 22″ × $\frac{3}{8}$″ cover plate. The vertical posts are made of two channels laced. The horizontal and vertical 4-angle struts are used to shorten the lengths of the posts and top chord. The lower chords and the diagonals are made of eye-bars. The floorbeams (not shown) are built-up plate girders 24 inches deep. The flanges are made of two 5″ × 3″ × $\frac{3}{8}$″ angles, while the web is $\frac{3}{8}$ inches thick. The joists are 9″ @ 21 lb. I beams for the main roadway, and 8″ Is and [s for the 6-ft. sidewalk.

The stress diagram for a 406-ft. span pin-connected Petit truss highway bridge is shown in Fig. 168, while the detail drawings are shown in Fig. 169*a* and Fig. 169*b*. The end-posts and top chords are

Bottom Lateral Bracing

Top Lateral Bracing.

FIG. 168. STRESS SHEET FOR 406-FT. SPAN PETIT TRUSS HIGHWAY BRIDGE.

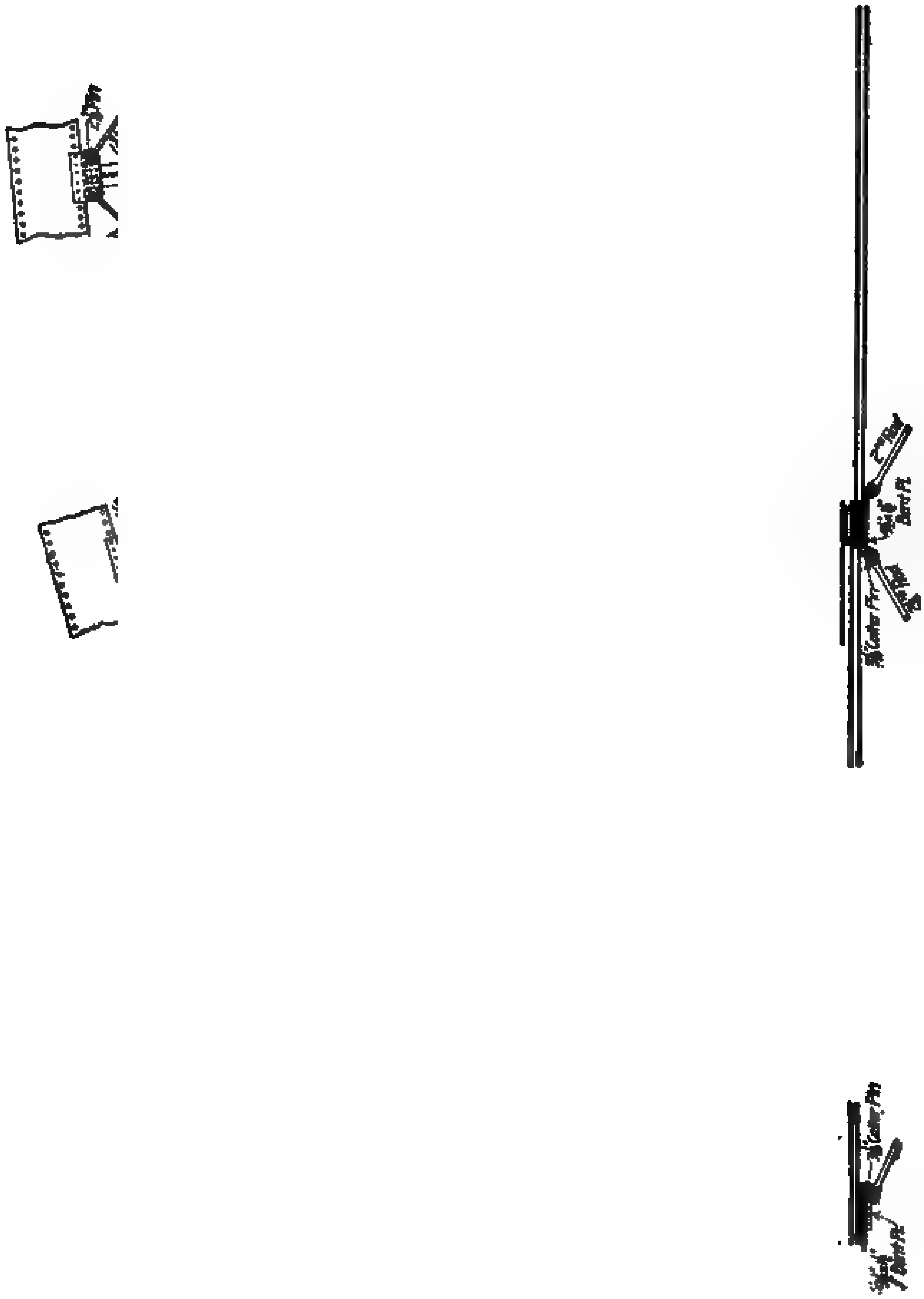


FIG. 169*a*. PETIT TRUSS HIGHWAY BRIDGE, 406-FT. SPAN.

Half Transverse Section.

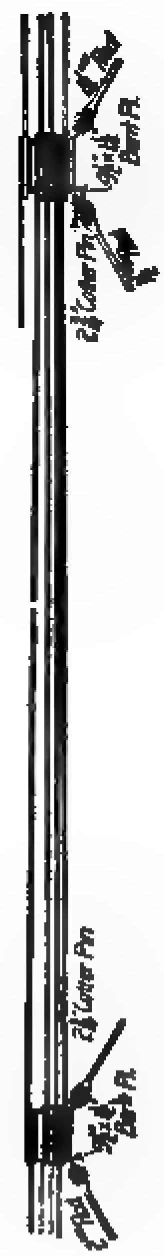


Fig. 169b. Petit Truss Highway Bridge, 406-ft. Span.

made of four angles and three plates, two plates on the sides and one plate on the top. The vertical posts are made of two channels laced, while the lower chords and the diagonals are made of eye-bars. The top and bottom laterals and the sway diagonals are made of adjustable rods. The floorbeams are built-up plate girders and are placed above the lower chords. The joists are 20″ I beams carried on shelf angles riveted to the webs of the floorbeams as shown. The floor is composed of buckle plates covered with 2½ in. of concrete, a one-inch layer of sand and a layer of 3″ asphalt blocks. The details of the buckle plates are shown in Fig. 189. The sidewalk floor is made of 2 in. of concrete carried on a plank floor, as shown in Fig. 188. The expansion end of the bridge is carried on two nests of expansion rollers as shown in Fig. 195.

Economic Depth and Panel Length of Trusses.—The economic depth and panel length of trusses is not capable of mathematical calculation. The minimum depth is determined by the required clear head room, which varies from 12½ to 15 feet. Short panel lengths give heavy trusses and light floor systems; while long panels give light trusses and heavy floor systems. For ordinary conditions it is not economical to use panel lengths less than 15 feet for short spans nor more than 25 feet for long spans. The minimum depth for through spans is about 16 feet where the floorbeams are placed below the lower chords. To make a stiff structure, the depth should be sufficient to have the floorbeams above the lower chords and to permit of efficient portal and sway bracing. Experience has shown that the most oconomical conditions occur when the angle θ, the tangent of which is the panel length divided by the depth, is 40 degrees. The top chord points of bridges with inclined chords should be approximately on a parabola passing through the pin at the hip.

The American Bridge Company uses the depths and panel lengths in its highway bridge standards as given in Table XV.

TABLE XV.

DEPTHS AND PANEL LENGTHS OF THROUGH HIGHWAY BRIDGES USED BY AMERICAN BRIDGE COMPANY.

Type of Truss.	Span, Feet.	Number of Panels.	Ratio of Depth to Panel Length.
Pratt, riveted and pin-connected.	80 to 90	5	1.0
	96 to 126	6	1.0
	133 to 147	7	1.0
	152 to 168	8	1.1
Quadrangular, Warren riveted.	80 to 90	5	1.0
	90 to 114	6	1.0
	119 to 133	7	1.0
	135 to 152	8	1.1
Camel-back, pin-connected.	162 to 180	9	1.0, 1.159, 1.25, 1.29
	190 to 220	10	1.0, 1.238, 1.28, 1.43
Petit, pin-connected.	240 to 276	12	1.0, 1.397, 1.555, 1.714
	294 to 322	14	1.0, 1.36, 1.60, 1.84, 2.00

TABLE XVI.

DEPTHS AND PANEL LENGTHS OF THROUGH HIGHWAY BRIDGES USED BY THE GILLETTE-HERZOG MFG. CO.

Type of Truss.	Span, Feet.	Number of Panels.	Ratio of Depth to Panel Length.
Pratt, pin-connected.	80 to 90	5	1.0
	96 to 120	6	1.1
	112 to 140	7	1.2 to 1.1
	128 to 160	8	1.35 to 1.3
	144 to 162	9	1.5 to 1.45
Camel-back, pin-connected.	171	9	1.1, 1.3, 1.5
	180	9	1.0, 1.2, 1.4
	180	10	1.1, 1.28, 1.46, 1.64
	190 to 200	10	1.05, 1.24, 1.43, 1.62
Petit, pin-connected.	200	10	1.05, 1.24, 1.43, 1.62
	204	12	1.4, 1.6, 1.75, 1.9
	216 to 228	12	1.2, 1.55, 1.78, 1.9
	240	12	1.05, 1.5, 1.7, 1.9
	280	14	1.05, 1.5, 1.69, 1.88, 1.94, 2.0

CHAPTER XII.

Plate Girder Bridges.

Introduction.—A plate girder consists of a vertical steel or iron web plate to whose top and bottom edges are riveted horizontal pairs

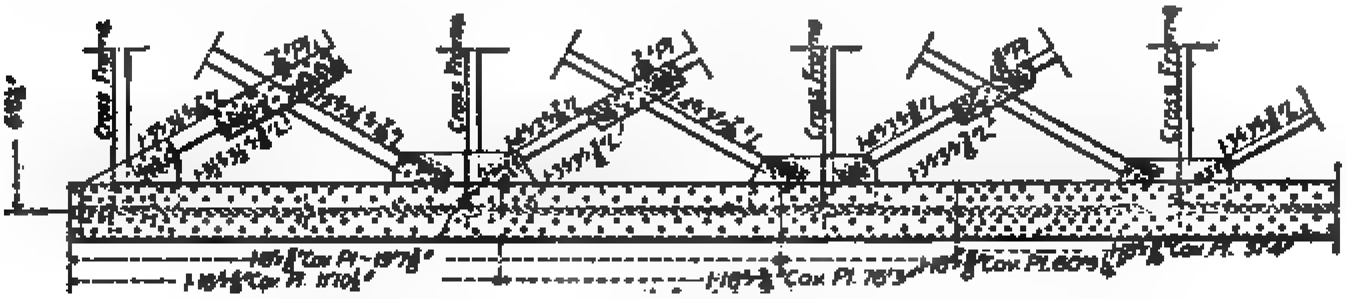


Fig. 170. Railway Deck Plate Girder Bridge.

of angles to form flanges, and to whose ends are attached vertical angles which transmit the load to the supports. Where the web plate is thin as compared with its depth, stiffener angles are riveted on opposite sides of the web, usually in pairs, at intervals not greater than

the depth of the girder, or five feet. Where the span is long, two or more plates are spliced together to form the web plates and horizontal plates are riveted to the flange angles to increase the flange area.

A plate girder bridge consists of two or more, usually two, plate girders fastened together by lateral bracing, and in the case of deck bridges by transverse bracing consisting of two or more cross-frames. In the railway deck plate girder bridge, Fig. 170, the roadway is carried

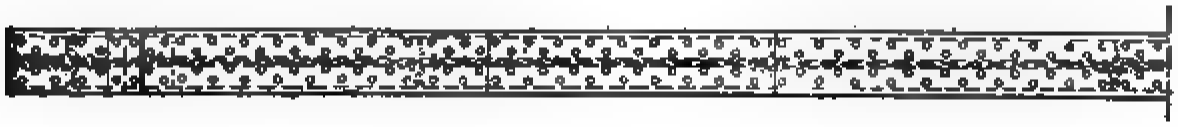


FIG. 171. RAILWAY THROUGH PLATE GIRDER BRIDGE.

directly on the tops of the girders. In a through plate girder bridge the roadway is carried on a floor system supported near the bottoms of the plate girders. A through plate girder railroad bridge is shown in Fig. 171, while a through plate girder highway bridge is shown in Fig. 172. As a through plate girder bridge can have only a lower lateral system, the upper flanges are braced by side braces with gusset plate connections.

Short spans up to 70 or 80 feet have one end fixed while the other end is allowed to move on a sliding plate. For greater lengths of span the expansion end is supported on nests of rollers.

The ordinary limit of plate girder spans is about 100 feet, although railroad plate girders having a span of 126 feet have been built. Plate girders of more than 100 feet span have a depth that makes transportation by rail very difficult.

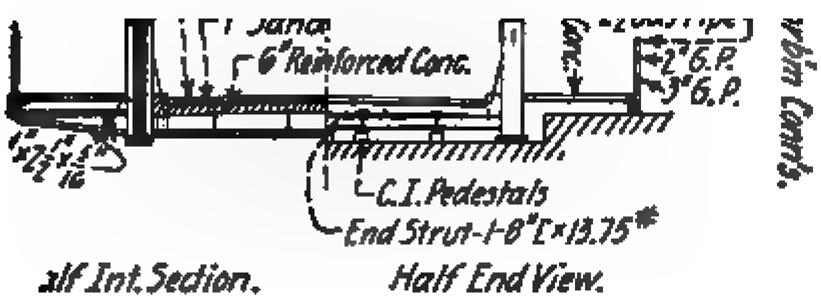


FIG. 172. HIGHWAY THROUGH PLATE GIRDER BRIDGE WITH SOLID FLOOR. (AMERICAN BRIDGE COMPANY.)

Thickness of Web.—Standard specifications limit the minimum thickness of the web plates to $\frac{3}{8}$ inch for railroad bridges and $\frac{5}{16}$ inch for highway bridges. For heavy loads and long spans the web plates are made much thicker than the minimum thickness. Thin webs require more stiffeners and give a much shorter life to the bridge.

Flanges.—The simplest form of a flange consists of a pair of unequal-legged angles with the long legs placed out and riveted to the web plate. When additional rivets are required in the connection of the flanges to the web plate, equal-legged angles with two rows of rivets are used. When additional area is required, one or more cover plates are usually riveted to the horizontal legs of the angles. The thickness of the flanges should be limited so that the rivets will not be longer than five times the diameter of the rivet. Flange angles should never be thinner than the web plates to which they are fastened. Where more than one plate is used, one plate should extend the full length of the girder, the others being continued a short distance (not less than one foot) beyond the point where the area is required. For steam or electric railway plate girder bridges the rivet heads and the variation in the thickness of the flange plates makes it necessary to notch the cross-ties unequally, so that other forms of flange are sometimes used for the upper flanges of long girders. It is quite the common practice to design the tension, or bottom, flange to take the stresses and then make the compression, or upper, flange with the same gross area.

Moments and Shears.—The moments and the shears in through plate girder bridges, as in Fig. 171 and Fig. 172, are found in the panels in the same manner as for a truss. In a deck bridge the moments and the shears are calculated in a similar manner, at intervals. The load on the girder produces shearing stresses in the girder, which in turn develop tensile and compressive stresses. In a solid rolled beam the entire section carries both shear and bending moment. In plate girders it is usual to assume that all the shear is carried by the web and that all the bending moment is taken by the flanges. The actual distribution of the moments and shears is about as follows:

Let F = the area of one flange, not including the included web plate;

h = height of girder between centers of gravity of the flanges in inches;

t = thickness of the web plate in inches;
f = allowable stress in flanges in lbs. per sq. in.;
$A = t \cdot h$ = area of web plate in square inches.

Then the total resisting moment of the girder is

$$M = F \cdot f \cdot h + f \cdot t \cdot h^2/6 \tag{80}$$

$$= F \cdot f \cdot h + A \cdot f \cdot h/6 \tag{81}$$

$$= (F + A/6) f \cdot h \tag{82}$$

This shows that approximately one-sixth of the web is available as flange area. On account of the reduction of the area due to rivet holes, one-eighth of the area of the web is commonly taken as available as flange area wherever this method is used.

Flange Area.—Assuming that all the bending moment is carried by the flanges, we have from (a) Fig. 173

$$M = F \cdot h \cdot f \tag{83}$$

where M is the bending moment in inch-pounds, F is the net area of the tension flange in square inches, f is the allowable unit stress in pounds, and h is the distance between centers of gravity of the flanges.

If one-eighth of the web area is considered available as flange area, we have

$$M = (F + A/8) h \cdot f \tag{84}$$

The rivet holes are to be deducted from the tension flange in calculating the net area. In calculating the net section of riveted tension members the holes should be taken as $\frac{1}{8}$ inch larger than the nominal size of the rivet. The compression flange is made with the same gross area as the tension flange.

Rivets in Flanges.—The loads produce shearing stresses in the web, which are transferred to the flanges by means of the rivets in the flanges. In (c) Fig. 173, let p be the pitch of the flange rivets, S be the vertical shear on the section, h be the distance between lines of rivets, and r be the allowable resistance of one rivet in pounds; then

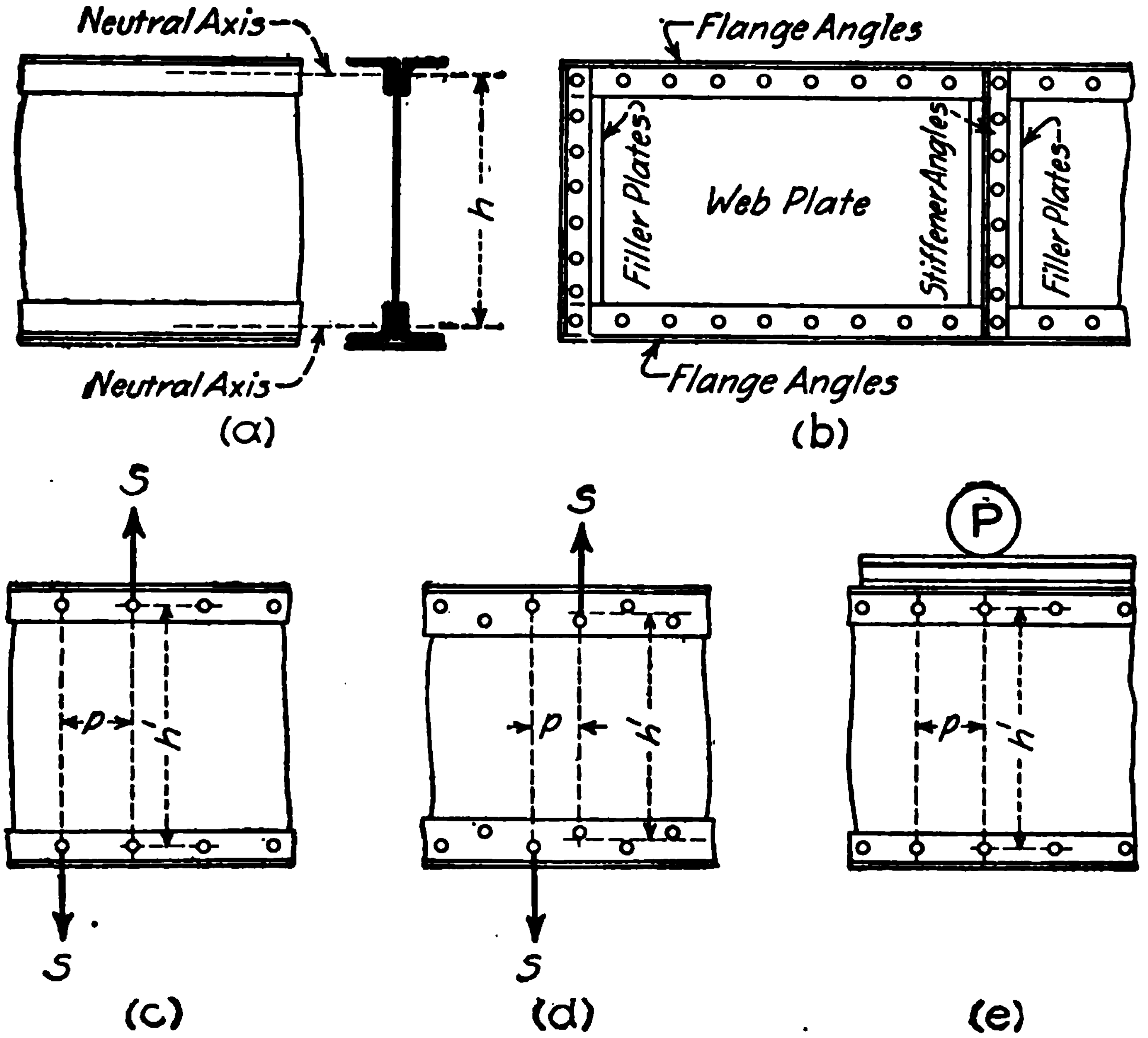


FIG. 173.

taking moments about the lower right-hand rivet we have, where the flanges carry all the bending moment

$$S \cdot p = r \cdot h, \text{ or } p = r \cdot h/S \tag{85}$$

If one-eighth of the web area is considered as available as flange area, the proportion of the shear producing flange stress will be

$$S' = S\frac{F}{F + A/8}, \text{ and } p = r \cdot h/S' = \frac{F + A/8}{F} r \cdot h/S \tag{86}$$

where F is the net area of one flange and A is the area of the web as before.

Rivets in Flanges Carrying Concentrated Loads.—In (e) Fig. 173 the rivets carry a shear due to the load P, in addition to the shear

due to the bending moment. If the rail distributes the load over a distance d, usually taken as the space covered by three ties or 42 inches, then the load per lineal inch will be $w = P/d$, and the vertical shearing stress on each rivet will be

$$r = \sqrt{(p \cdot w)^2 + (S \cdot p/h)^2}$$

and

$$p = \frac{r}{\sqrt{w^2 + (S/h)^2}} \tag{87}$$

If the web is assumed to resist one-eighth of the bending moment, substitute S' as given in (86) for S in (87).

Web Splice.—In Fig. 174 if the flanges are assumed to take all of the bending moment and the web plate to take all of the shear, the rivets on one side of the splice will take the shearing stress on the section, and in addition will have a shearing stress due to the moment $M' = S$ times the distance between the centers of gravity of the rivets on each side. The latter stress is usually very small and may ordinarily be neglected. If r is the stress on one rivet, $2n$ the number of rivets on one side, and S the vertical shear, then

$$r = S/2n \tag{88}$$

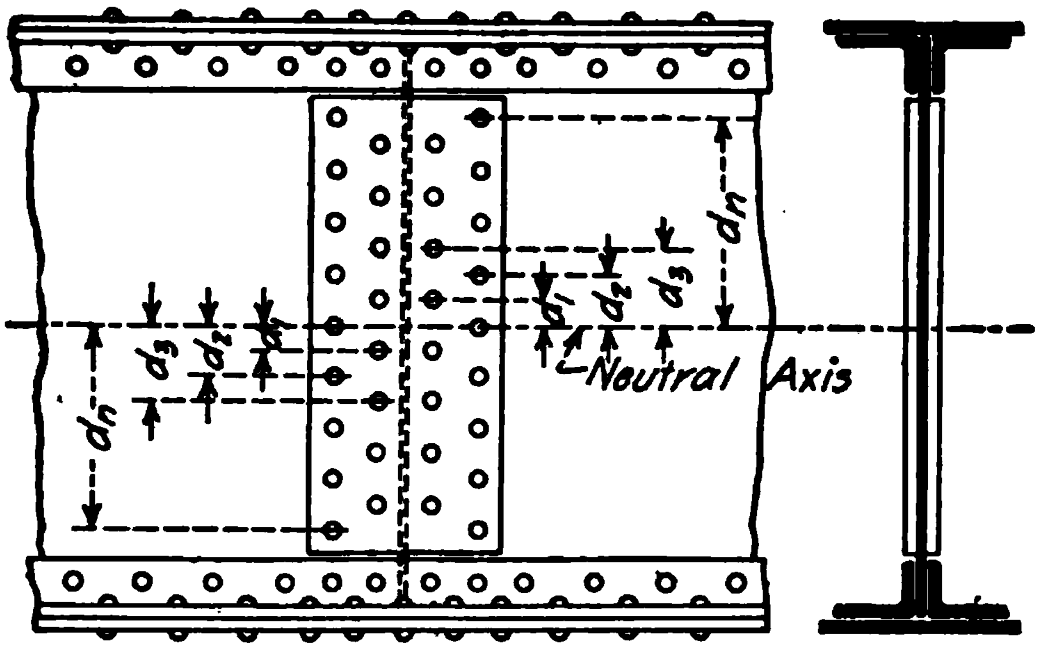


FIG. 174.

If the web is assumed to carry one-eighth of the bending moment in addition to the shear, the rivets will in addition resist this bending moment with lever arms d_1, d_2, etc., Fig. 174. The stresses due to this

moment will vary as the distance of the rivet from the neutral axis, while the resistance of the rivet will vary as the square of the distance from the netral axis. Now if a is the stress on a rivet due to bending

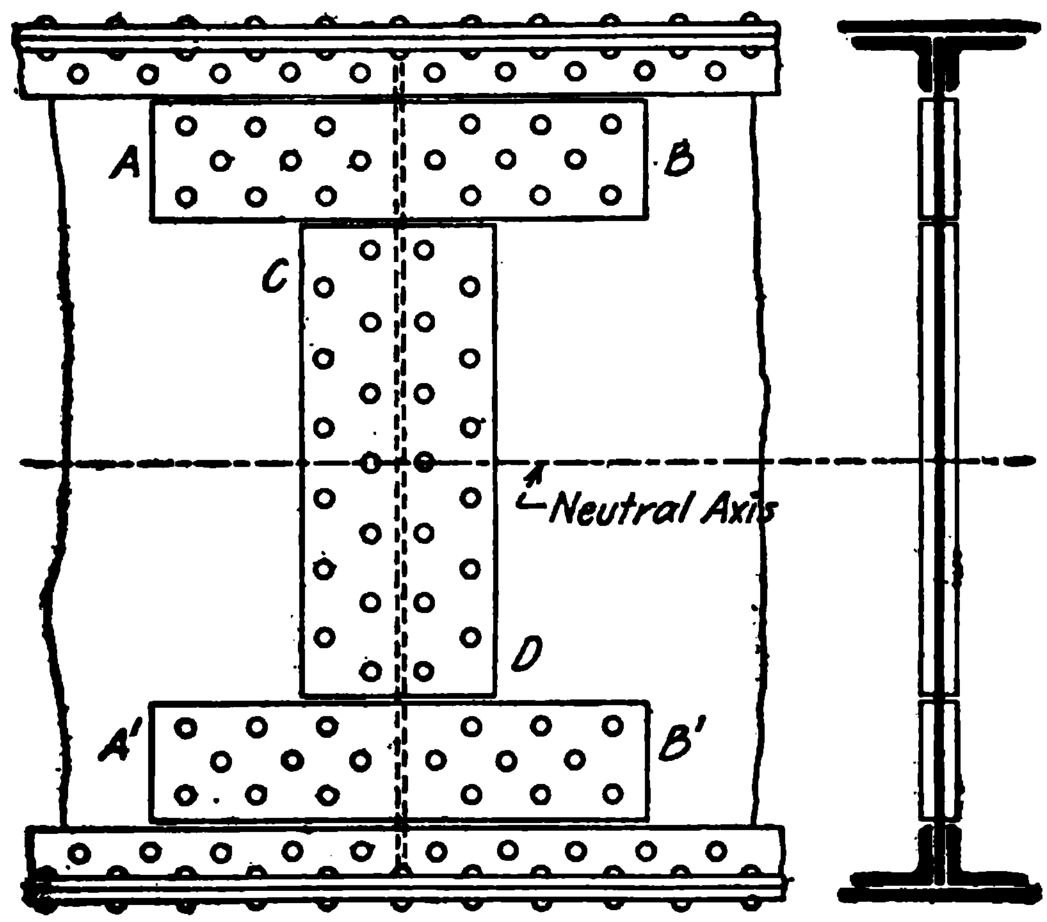


FIG. 175.

at a unit's distance from the neutral axis, the stress at a distance d_1 will be $a \cdot d_1$; at a distance d_2 will be $a \cdot d_2$; etc. The total resistance of the rivets on one side of the splice will be $2a(d_1^2 + d_2^2 + d_3^2 \cdots + d_n^2) = M'$, where M' is one-eighth of the total bending moment.

Having calculated a, the horizontal shear on the rivet at a distance d_1 from the neutral axis is $r_m = a \cdot d_1$, and the total shear on the rivet will be

$$r = \sqrt{r_s^2 + r_m^2} \qquad (89)$$

Where the splice plates are designed to transfer the bending moment as well as the shear, the splice shown in Fig. 175 is sometimes used. Plates AB and $A'B'$ are assumed to transfer all the bending moment, and plate CD to transfer all the shear.

Flange Splices.—The flange angles are spliced with short pieces of angles. Both flange angles should not be spliced at the same point, but the splices should be separated by several feet.

Design of Web Stiffeners.—Web stiffeners are used to prevent the buckling of the web plate, and are usually spaced somewhat less than the depth of the girder. There is no rational method for the design of stiffener angles. Tests show that the stiffener acts as a beam to prevent buckling of the web and is not appreciably stressed in the direction of its length, except where it is used at points of concentrated loading. A common specification is that "Stiffeners shall be used at distances not greater than the depth of the girder or five feet, where the actual shearing stress is greater than given by the formula—allowable shearing stress $= 12{,}500 - 90H$, where H is the ratio of the depth to the thickness of the web plate. Where stiffeners are required they shall be designed as columns with an allowable unit stress of $P = 12{,}000 - 55l/r$, where $l =$ the length and $r =$ the radius of gyration of the stiffener angles at right angles to the web plate, both in inches." "Stiffeners shall be provided at all points of concentrated loading, and shall contain enough rivets to transfer the vertical shear to the web plate."

Camber.—Plate girders are cambered by separating the web plates by the required amount in the upper part of the web splice. Plate girders in which a single web plate is used without a splice cannot be cambered. Many engineers do not camber plate girders.

Economical Depth.—Plate girders are commonly made with a depth of from $\frac{1}{8}$ to $\frac{1}{12}$ of the span. An approximate empirical rule for the depth is to make the area of the two flanges equal to the area of the web plate.

Example of Calculation.—The maximum shears and moments in an 86-foot span deck plate girder railway bridge are shown in Fig. 176. The shear at any point may be found by scaling from the shear curve to the horizontal base line, while the moments may be obtained by scaling from the moment curve to the base line. The composition of the flanges and the lengths of the plates are shown. The maximum shears and bending moments were calculated at intervals of about 7 feet, and the curves were drawn through these points.

Details of Plate Girders.—The general plans for an 80-foot through plate girder highway bridge are shown in Fig. 172. The roadway is 20 feet in the clear, with two 6½-ft. sidewalks. The floor of the roadway consists of 6 inches of reinforced concrete carried on 12-inch I beam joists, covered with a wearing surface of paving brick.

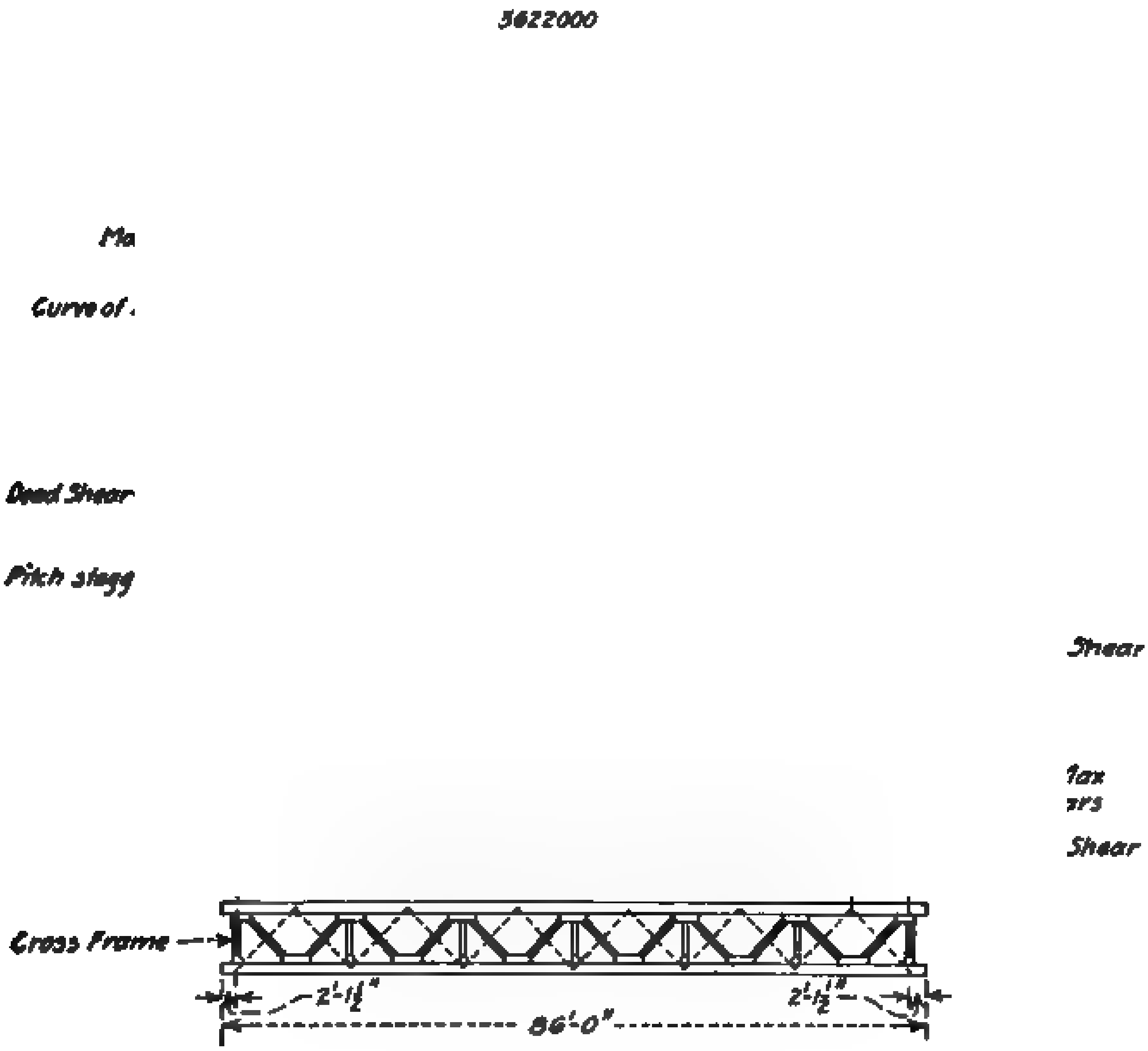


Fig. 176. Shears and Moments in a Railway Plate Girder.

The detailed shop plans of a 65-ft. deck plate girder electric railway bridge are shown in Fig. 177. This girder span was designed according to Cooper's 1901 Specifications for E_1 loading. The ties were carried directly on the top flanges of the girders. These details represent good practice in light plate girder construction. It will be noted that the stiffener angles are crimped to go over the flange angles

except where two hitch stiffener angles are used for cross-frame connections, in which case fillers are used. The ties were fastened to the upper flange angles by means of hook bolts in every third tie, while the 6×8 inch guards were bolted to the ties. The upper and lower laterals are of the Warren type and are made of single angles. Four intermediate and two end cross-frames are used. This bridge was designed by the author and was fabricated by the American Bridge Company.

CHAPTER XIII.

Design of Truss Members.

KINDS OF STRESS.—In addition to the stresses due to (1) dead load, (2) live or moving load, (3) wind load, and (4) snow load considered in Part I, it will be necessary to consider the following stresses: (5) impact stresses; (6) temperature stresses; (7) centrifugal stresses, and (8) secondary stresses not taken into account in the calculations. In addition to the above it is necessary in determining the allowable stress in any member to take into account imperfections in materials and workmanship, possible increase in live loads, fatigue of metals, the frequency of the application of the stress, corrosion and deterioration of materials, etc. The structure should be so designed that no part will be ever stressed beyond the elastic limit. The allowable stresses for dead load are usually taken at about 60 to 70 per cent of the elastic limit of the material. For example in the case of steel with an elastic limit of 30,000 lbs., the allowable working stresses for dead loads alone would vary from 18,000 to 21,000 lbs. per sq. in.

Impact Stresses.—As a load moves over the bridge it causes shocks and vibrations whereby the actual stresses are much increased over those due to static loads alone. It is shown in mechanics of materials that a load suddenly applied to a bar or a beam will produce stresses twice the stresses produced by the same load gradually applied. A bridge is a complex structure and it is not possible to determine the exact effect of the moving loads. It has been found by experiment that the ultimate strength for repeated loads is much less than the ordinary ultimate strength. In a bridge it will be seen that the dead load is a fixed load and that the live load is a varying load. Impact stresses appear to be intimately connected with the fatigue of metals.

For stresses of one kind Professor Launhardt has proposed the following formula:

$$P = S\left(1 + \frac{\text{Min. stress}}{\text{Max. stress}}\right) \tag{90}$$

where P is the allowable working stress required, and S is the allowable working stress for live loads, varying from zero to the maximum stress. For stresses of opposite kinds Professor Weyrauch has proposed the following formula:

$$P = S\left(1 - \frac{\text{Min. stress}}{2\ \text{Max. stress}}\right) \tag{91}$$

where P and S are the same as for the Launhardt formula, the maximum and minimum stresses being taken without sign. For columns and struts the allowable stresses are to be reduced by a suitable column formula.

There are three methods in common use for taking account of impact and fatigue: (1) Impact formulas; (2) Launhardt-Weyrauch formulas, and (3) Cooper's Method.

(1) *Impact Formulas.*—The formula in most common use is given in the form

$$I = S\left(\frac{a}{L + b}\right) \tag{92}$$

where I = impact stress to be added to the static live load stress; S = the static live load stress, L = the length in feet of the portion of the bridge that is loaded to produce the maximum stress in the member, and a and b are constants expressed in feet. The American Bridge Company and the American Railway Engineering and Maintenance of Way Association specify for railway bridges, $a = b = 300$ ft. Mr. J. A. L. Waddell specifies $a = 400$ ft., and $b = 500$ ft. for railway bridges; and $a = 100$ ft., and $b = 150$ ft. for highway bridges.

For highway bridges the American Bridge Company specifies that the maximum live load stress shall be increased 25 per cent to cover impact and vibration.

Mr. C. C. Schneider, M. Am. Soc. C. E., specifies that for electric railway bridges

$$I = S \cdot 150/(L + 300) \tag{92a}$$

In the Osborn Engineering Company's 1901 specifications for railway and for highway bridges the impact is calculated by the formula

$$I = S \cdot S/(S + D) \tag{92b}$$

where S is the static live load stress and D is the dead load stress. This method is also specified by the Harriman Railway System.

(2) *Launhardt-Weyrauch Formulas.*—Formula (90) is used for determining the allowable stress for loads of one kind and formula (91) is used for determining the allowable stress for loads of different kinds. This method is used in Thacher's Specifications, and others.

(3) *Cooper's Method.*—Cooper uses formula (90) and calculates the area for the dead load and the area for the live load stress separately. For dead loads from formula (90) we have $P = 2S$, while for live loads the range of stress is from zero to the maximum, and $P = S$.

For a reversal of stress Cooper designs the member to take both kinds of stress, but to each stress he adds eight-tenths of the lesser of the two stresses.

The different methods, while apparently very unlike, give essentially the same results. Many engineers claim that fatigue of the materials and impact should be considered separately. In choosing working stresses, an allowance should be made for secondary and other stresses that cannot easily be calculated, for corrosion, etc.

Temperature Stresses.—An increase or decrease in temperature produces no stresses in a bridge with one end on frictionless rollers. Where there is a horizontal resistance to the movement, the bridge becomes a two-hinged arch and it is necessary to calculate the stresses for that case. See the author's "Steel Mill Buildings," Chapter XIV.

Centrifugal Stresses.—Mr. C. C. Schneider's "Specifications for Electric Railway Bridges" contains the following requirement:

Structures located on curves shall be designed for the centrifugal force of the live load acting at the top of the rail. The centrifugal force shall be calculated by the following formula:

$$C = (0.043 - 0.003D)W \cdot D \tag{93}$$

where

$C =$ centrifugal force in lbs.;
$W =$ weight of train in lbs.;
$D =$ degree of curvature.

SPECIFICATIONS FOR STEEL.—All standard specifications call for Open Hearth steel. It has been the custom to specify "soft" steel with an ultimate strength of, say, 54,000 to 62,000 lbs. per sq. in.; "medium" steel with an ultimate strength of, say, 60,000 to 68,000 lbs. per sq. in.; and "rivet" steel with an ultimate strength of, say, 50,000 to 58,000 lbs. per sq. in. The American Railway Engineering and Maintenance of Way Association has specified a single grade of "structural" steel with an ultimate strength of 56,000 to 64,000 lbs. per sq. in.; and "rivet" steel with an ultimate strength of 46,000 to 54,000 lbs. per sq. in. This method appears to be coming rapidly into use and promises to become standard.

Standard specifications are given in Appendix I, in which the specifications for material adopted by the American Railway Engineering and Maintenance of Way Association have been used.

PERMISSIBLE STRESSES.—The allowable stresses in the different members of steel highway bridges will depend upon the method of providing for impact.

Schneider's Specifications.—In his "Specifications for Steel Electric Railway Bridges" Mr. C. C. Schneider has specified the allowable stresses adopted by the American Railway Engineering and Maintenance of Way Association, using the impact formula in (92a). The clauses referring to unit stresses are as follows:

§ 17. *Unit Stresses.*—All parts of structures shall be so proportioned that the sum of the maximum stresses shall not exceed the following amounts in pounds per sq. in., except as modified in paragraphs 25 to 27.

§ 18. *Tension.*—Axial tension on net section........................16,000

§ 19. *Compression.*—Axial compression on gross section......$16{,}000-70 \cdot l/r$ where "l" is the length of member in inches and "r" is the least radius of gyration in inches.

§ 20. *Bending:* on extreme fibers of rolled shapes, built sections and girders; net section..16,000
on extreme fibers of pins..................................24,000

§ 21. *Shearing:* shop driven rivets and pins........................12,000
field driven rivets and turned bolts........................10,000

plate girder webs; gross section.......................... 10,000

§ 22. *Bearing:* shop driven rivets and pins.......................... 24,000
field driven rivets and turned bolts.......................... 20,000
granite masonry and Portland cement concrete............... 600
sandstone and limestone.................................... 400
expansion rollers; per linear inch.......................... 600 *d*
where "*d*" is the diameter of the roller in inches.

§ 23. *Limiting Length of Compression Members.*—No compression member shall have a length exceeding 100 times its least radius of gyration, excepting those for wind bracing, which may have a length 120 times the least radius of gyration.

§ 24. *Alternate Stresses.*—Members subject to alternate stresses of tension and compression shall be proportioned for the stresses giving the largest section. If the alternate stresses occur in succession during the passage of one train, as in stiff counters, each stress shall be increased by 50 per cent of the smaller. The connections shall in all cases be proportioned for the sum of the stresses.

§ 25. *Counters.*—Wherever the live and dead load stresses are of opposite character, only 70 per cent of the dead load stress shall be considered as effective in counteracting the live load stress.

§ 26. *Combined Stresses.*—Members subject to the action of both axial and bending stresses shall be proportioned so that the combined stress shall not exceed the allowed axial stress.

§ 27. *Lateral and Other Stresses Combined.*—For stresses produced by lateral and wind forces combined with those of live loads, dead loads and centrifugal forces, the unit stresses may be increased 25 per cent over those given above; but the section shall not be less than required if lateral and wind forces be neglected.

American Bridge Company's Specifications.—The American Bridge Company's 1901 "Specifications for Steel Highway Bridges" specify the following allowable unit stresses. Impact equal to twenty-five per cent of the live load included.

Tension.—Axial tension on net section, soft steel 15,000 lbs. per sq. in., medium steel 17,000 lbs. per sq. in.

Compression.—For soft steel

$$P=\frac{15{,}000}{1+l^2/13{,}500r^2}$$

For medium steel.

$$P=\frac{17{,}000}{1+l^2/11{,}000r^2}$$

where P = permissible working stress per sq. in. in compression, l = length of member in inches, c to c of connection; r = least radius of gyration of the section in inches.

Length of Compression Members.—Main members shall not have the length greater than 120 times the least radius of gyration, and lateral struts not greater than 140 times the least radius of gyration.

Reversal of Stress.—In class A, B, C and D members subject to the reversal of stress shall be designed to take either stress. In classes E_1 and E_2 members subject to reversal of stress shall be designed so that the total area shall be equal to the sum of the areas required for the tensile and compressive stresses.

Combined Stress.—The allowable stresses when wind stresses are added to the stresses due to vertical loads and impact, shall not exceed 19,000 lbs. per sq. in. for soft steel, nor 21,000 lbs. per sq. in. for medium steel.

Bending.—The bending on rolled shapes and plate girders, net section, shall not exceed 15,000 lbs. per sq. in. for soft steel, nor 17,000 per sq. in. for medium steel. Bending on extreme fibers of pins shall not exceed 22,000 lbs. per sq. in. for soft steel, nor 25,000 lbs. per sq. in. for medium steel.

Shear.—The shear on shop rivets shall not exceed 11,000 lbs. per sq. in. for soft steel, nor 12,000 lbs. per sq. in. for medium steel; the number of field rivets shall be increased 25 per cent.

Allowable shear in webs of plate girders shall not exceed 9,000 lbs. per sq. in. for soft steel, nor 10,000 lbs. per sq. in. for medium steel; it being assumed that $\frac{1}{8}$ of the web area is available as flange area.

Bearing.—The bearing on rivets and pins shall not exceed 22,000 lbs. per sq. in. for soft steel, nor 24,000 lbs. per sq. in. for medium steel.

Bearing on masonry shall not exceed 400 lbs. per sq. in.

Bearing on rollers shall not exceed $1{,}200\sqrt{d}$ lbs. per lineal inch, where $d =$ diameter of roller in inches.

Cooper's Specifications.—In his 1901 "Specifications for Steel Highway and Electric Railway Bridges," Mr. Theodore Cooper specifies the following allowable unit stresses:

Tension. Medium Steel.—Floorbeam hangers and other similar members liable to sudden loading, net section, 8,000 lbs. per sq. in.; longitudinal, lateral and sway bracing for wind and live load stresses, 18,000 lbs. per sq. in.; solid rolled beams used as cross floorbeams and stringers, 13,000 lbs. per sq. in.; bottom flanges of riveted girders, net section, all moment resisted by flanges, 13,000 lbs. per sq. in.; bottom chords, main diagonals, counters and long verticals, 12,500 lbs. per sq. in. for live load stress, and 25,000 lbs. per sq. in. for dead load stress.

Soft steel may be used with tensile unit stresses 10 per cent less than above

Compression. Medium Steel.—For chord segments $P = 12{,}000 - 55 \cdot l/r$ lbs. per sq. in. for live load stresses, and $P = 24{,}000 - 110 \cdot l/r$ lbs. per sq. in. for dead load stresses.

For all posts for through bridges, including end-posts, $P = 10{,}000 - 45 \cdot l/r$ lbs. per sq. in. for live load stresses, and $P = 20{,}000 - 90 \cdot l/r$ lbs. per sq. in. for dead load stresses.

For all posts in deck bridges and trestles, $P = 11{,}000 - 40 \cdot l/r$ lbs. per sq. in. for live load stresses, and $P = 22{,}000 - 80 \cdot l/r$ lbs. per sq. in. for dead load stresses.

For lateral struts and rigid bracing $P = 13{,}000 - 60l/r$ lbs. per sq. in. for wind stresses, and $\frac{2}{3}$ of the above for live load stresses.

In above $l =$ length of member c to c of connections, and $r =$ radius of

gyration of the member, both in inches. The ratio of l/r shall not exceed 100 for main members and 120 for laterals.

Soft steel may be used with unit stresses 15 per cent less than the above.

Bending.—The bending on extreme fibers of pins shall not exceed 20,000 lbs. per sq. in.

Shear.—The shear on pins and rivets in trusses shall not exceed 10,000 lbs. per sq. in. The shear on rivets in floor systems shall not exceed 80 per cent, and in lateral systems shall not exceed 140 per cent of the values above.

Bearing.—The bearing on pins and rivets in trusses shall not exceed 18,000 lbs. per sq. in. The bearing on rivets in floor systems shall not exceed 80 per cent, and in lateral systems shall not exceed 140 per cent of the values above.

Field rivets shall have their allowable bearing and shear reduced one-third

Bearing on rollers per lineal inch $= 300d$, where d is the diameter of the roller in inches.

Bearing on masonry shall not exceed 275 pounds per sq. in.

Combined Stresses.—Members subject to combined stresses must be designed for the greatest stress. Unless the stress due to weight, only, exceed 10 per cent of the allowed stress, such stress need not be considered. Unless the stress due to wind forces exceed 25 per cent of the allowed unit stress it need not be considered.

Reversal of Stress.—Members and their connections subject to alternate stress shall be designed to take each kind of stress. Both stresses, shall, however, be increased by an amount equal to $\frac{8}{10}$ of the least of the two stresses.

Osborn Engineering Company's Specifications. — The Osborn Engineering Company's 1901 "Specifications for Steel Highway Bridges" specify the following allowable unit stresses. Impact as calculated by (92b) is included.

Tension.—Wrought iron 18,000 lbs. per sq. in.; soft steel 20,000 lbs. per sq. in.; medium steel 22,000 lbs. per sq. in.; net section.

Compression.— Compression members shall be designed for a unit stress not greater than given by the formula

$$P = \frac{C}{1 + l^2/B \cdot r^2} \text{ lbs. per sq. in.}$$

in which $C = 18{,}000$ for wrought iron, $= 20{,}000$ for soft steel, $= 22{,}000$ for medium steel; $B = 36{,}000$ for members with two square bearings, $= 24{,}000$ for members with one square and one pin bearing, $= 18{,}000$ for members with two pin bearings; $l =$ length of members between supports and $r =$ least radius of gyration of the member, both in inches. The ratio of l/r shall not exceed 125 for main members nor 150 for subordinate members.

Bending.—Bending on medium steel pins shall not exceed 25,000 lbs. per sq. in.

Bearing.—Bearing on medium steel pins shall not exceed 22,000 lbs. per sq. in., on rivets not greater than 20,000 lbs. per sq. in.

Bearing on masonry shall not exceed 400 lbs. per sq. in.

Bearing on rollers shall not exceed $600d$ lbs. per lineal inch of roller, where d = diameter of roller in inches.

Shear.—The allowable shear on pins and rivets shall not exceed 10,000 lbs. per sq. in.

Combined Stresses.—Members subjected to combined, direct and bending stresses shall be proportioned for the combined stresses.

Reversal of Stress.—Members subjected to alternate tensile and compressive stresses shall be designed to resist either, and shall have 25 per cent excess of strength in their connections.

For additional specifications, see Appendix I.

THICKNESS OF METAL.—The American Bridge Company specifies that no metal less than $\frac{1}{4}$ in. thick shall be used for highway bridges, except for fillers. Cooper, Schneider and Osborn specify that no metal less than $\frac{5}{16}$ in. thick shall be used for highway bridges, except for fillers. The author recommends that $\frac{1}{4}$ in. metal be permitted in class D highway bridges, and that $\frac{5}{16}$ in. metal be the minimum for all other highway bridges.

The standard minimum thickness for metal in railway bridges is $\frac{3}{8}$ in.

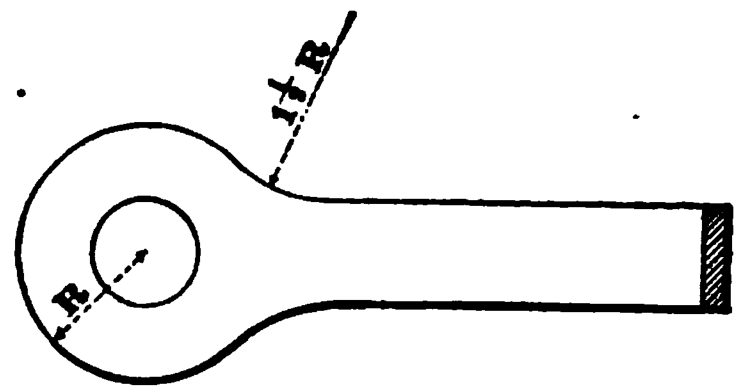
TENSION MEMBERS.—Tension members are made (1) of eye-bars; (2) of square or round loop bars; (3) of simple shapes, and (4) of built sections.

Eye-bars.—Eye-bars are used for main tension members of pin-connected trusses. The eyes may be formed (*a*) by upsetting and forging, or (*b*) by piling and welding. By the first method the bar is upset and the head is forged in a die, after which the bar is reheated and annealed and the pin hole is drilled. By the second method a "pile" of iron bars is placed on the end of the bar, the pile is heated and the head is forged in a die. The bar is then reheated and annealed and the pin hole is drilled. Steel eye-bars should always be made by upsetting and forging. The American Bridge Company's standard eye-bars are given in Table XVII; while the King Bridge Company's standard eye-bars are given in Table XVIII. Eye-bars thinner than those specified are liable to buckle in the head. Eye-bars may be obtained in different thicknesses varying by $\frac{1}{16}$ inch. Eye-bars are seldom made with a thickness of more than one-third or less than one-sixth of the depth of the bar. The Osborn Engineering Company specifies that bars shall not be less than $\frac{5}{8}$ in. in thickness, and prefer-

ably not less in thickness than $\frac{1}{8}$ the depth. Eye-bars should be parallel as nearly as possible; the B. & O. R. R. specifies that eye-bars shall not be out of line more than one inch in ten feet; the specifications in Appendix I require that eye-bars shall not be out of line more than one inch in 16 feet (§ 92). Thick bars give large moments on the pin. Pins are ordinarily specified to be not less than three-fourths of the

TABLE XVII.

FORGED EYE-BARS. AMERICAN BRIDGE COMPANY STANDARDS.



Bar.		Head.			Bar.		Head.		
Width. Ins.	Minimum Thickness. Ins.	Diameter. Ins.	Maximum Pin. Ins.	Additional Material for One Head. Ft. and Ins.	Width. Ins.	Minimum Thickness. Ins.	Diameter. Ins.	Maximum Pin. Ins.	Additional Material for One Head. Ft. and Ins.
		4½	1¾	1- 0		1	16½	7	2- 7
2	½	5½	2¾	1- 4	7	1⅛	17½	8	2-11
		*6½	3¾	1- 9		1⅛	*18½	9	3- 4
		6	2½	1- 3		1	18	7	2- 8
2½	⅝	7	3½	1- 7	8	1⅛	19	8	3- 0
		*8	4½	2- 0		1¼	*20	9	3- 4
		7½	3¼	1- 6		1⅛	22½	9	3- 5
3	⅝	8½	4¼	1-11	10	1¼	24	10½	3- 9
		*9½	5¼	2- 4		1⅜	*25	11½	4- 1
	¾	10	4½	1-11		1¼	26½	10	3- 8
4	⅞	11	5½	2- 3	12	1⅜	28	11½	4- 2
	1	*12	6½	2- 8		1½	*29½	13	4- 8
	¾	12	5¼	2- 1		1⅜	31	12	4- 3
5	1	13½	6¾	2- 8	14	1½	33	14	4-10
	1	*15	8¼	3- 3		1⅝	*34	15	5- 5
	¾	14	5¾	2- 4		1¾	36	14	4-11
6	1	14¾	6½	2- 6	16	1⅞	*37½	16	5- 5
	1	*16½	8¼	3- 2					

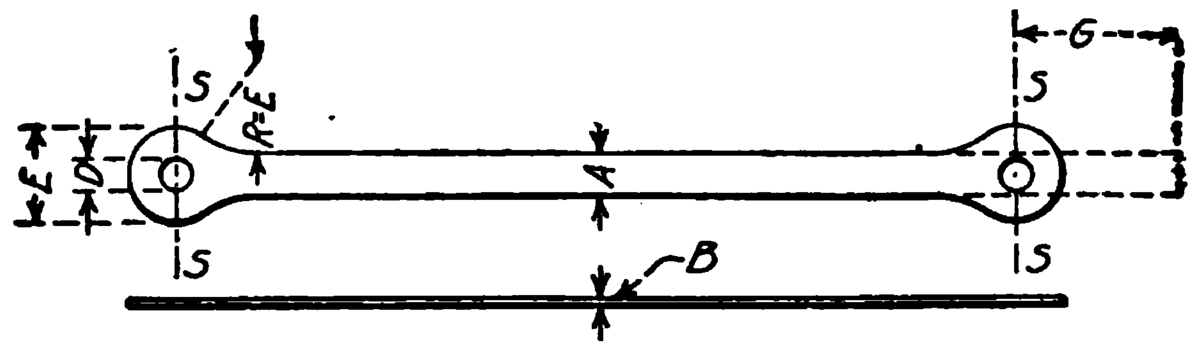
Bars marked* should only be used when absolutely unavoidable.

Bars are hydraulic forged and are guaranteed to develop the full strength of the bar under conditions given in the table when tested to destruction.

depth of the deepest bar coming on the pin. Bars very shallow or very deep will therefore require large pins. The stresses in eye-bars due to their own weight are given in Fig. 107, Chapter VIII. Eye-bars should always be used in pairs and should be kept small in order to keep down the size of the pins and reduce the cost of fabrication of the pins. Specifications for eye-bars are given in Appendix I.

TABLE XVIII.

FORGED EYE-BARS. KING BRIDGE COMPANY STANDARDS.



Bar.		Head.		
A	B	E	D	G
Width. Ins.	Minimum Thickness. Ins.	Diameter. Ins.	Maximum Pin. Ins.	Additional Material for One Head. Ft. and Ins.
1½	⅝	3¼	1⅝	0- 8
	⅝	6	3⅞	1- 6
2	¾	3¼	15/16	0- 7
	¾	5	2 3/16	0-11
	¾	6	3 3/16	1- 3
2½	¾	5	1½	0-10
	¾	6	2½	1- 0
	¾	7½	4	1- 5
3	¾	6	1 13/16	0-10¾
	¾	8	3 13/16	1- 4
	¾	10	5 13/16	1-11
3½	¾	7	2 1/16	0-11
	¾	8	3 1/16	1- 1
	¾	10	5 1/16	1- 8½

Bar.		Head.		
A	B	E	D	G
Width. Ins.	Minimum Thickness. Ins.	Diameter. Ins.	Maximum Pin. Ins.	Additional Material for One Head. Ft. and Ins.
4	¾	8	2⅜	1- 0
	¾	10	4⅜	1- 5
	¾	12	6⅜	2- 4½
5	1	11	4	1- 4½
	1	13	6	2- 1
	1	15	8	2- 8
6	1	13½	5⅛	1- 8½
	1	15	6⅝	2- 1
	1	16½	8⅛	2- 8
8	1⅜	18	6 13/16	2- 3
	1⅜	20	8 13/16	2-10½
10	1¾	20	6	2- 2
	1¾	24	10	3- 3
12	1¾	22	5 3/16	2- 1½
	1¾	24	7 7/16	2- 7

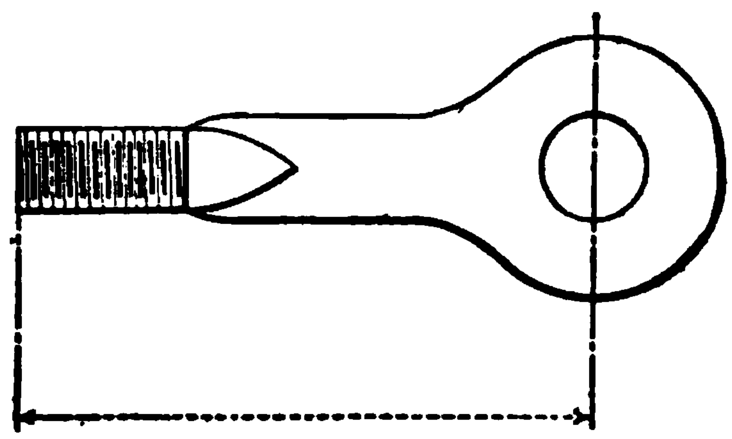
The area of the head on the line *S-S* is 140 per cent of the area of the bar.

Adjustable Eye-bars.—Where eye-bars are used for counters they are made adjustable. The American Bridge Company's standard

adjustable eye-bars are given in Table XIX. The parts of the bar may be connected by sleeve nuts or turnbuckles.

TABLE XIX.

ADJUSTABLE EYE-BARS. AMERICAN BRIDGE COMPANY STANDARDS.



Min. Length C. to end 6′ 6″ preferably

Bar.		Screw End.			Bar.		Screw End.		
Width. Ins.	Thickness. Ins.	Diameter. Ins.	Length. Ins.	Additional Material for Upset. Ins.	Width. Ins.	Thickness. Ins.	Diameter. Ins.	Length. Ins.	Additional Material for Upset. Ins.
2	*5/8	1 1/4	4	12	6	*1	3 1/2	7	12
	3/4	1 7/8	4 1/2	12		1 1/8	3 3/4	7	12
	7/8	2	4 1/2	11		1 1/4	4	7 1/2	13
2 1/2	*3/4	2 1/8	4 1/2	12		1 3/8	4 1/4	8	14
	7/8	2 1/4	5	12	7	*1 1/8	4	7 1/2	12
	1	2 3/8	5	12		1 1/4	4 1/4	8	13
3	*3/4	2 1/4	5	12		1 3/8	4 1/2	8 1/2	14
	7/8	2 1/4	5	10		1 1/2	4 3/4	8 1/2	14
	1	2 5/8	5 1/2	13	8	*1 1/8	4 1/4	8	12
4	*3/4	2 5/8	5 1/2	13		1 1/4	4 1/2	8 1/2	13
	7/8	2 3/4	5 1/2	11		1 3/8	4 3/4	8 1/2	13
	1	3	6	13		1 1/2	5	9	14
	1 1/8	3 1/4	6 1/2	14		1 5/8	5 1/4	9 1/2	15
5	*3/4	2 7/8	6	12					
	7/8	3	6	11					
	1	3 1/4	6 1/2	12					
	1 1/8	3 1/2	7	13					
	1 1/4	3 3/4	7	14					

Bars marked * should only be used when absolutely unavoidable.

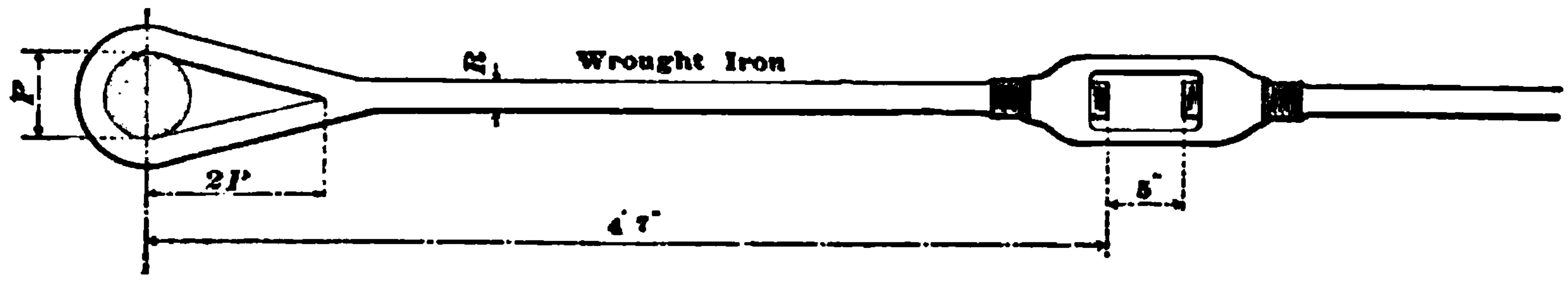
Bars are hydraulic forged and are guaranteed to develop the full strength of the bar under conditions given in the table when tested to destruction.

Loop-bars.—Iron bars, both square and round, are often made with loop ends. Steel bars should never be used with loop ends for the

TABLE XX.

LOOP-BARS. AMERICAN BRIDGE COMPANY STANDARDS.

All Dimensions in Inches.



Length in inches beyond pin centre to form one eye equals 3.7 $(P + R)$

DIAM. OF PINS.	DIAMETER OR SIDE OF BARS.												DIAM. OF PINS.
	3/4	7/8	1	1 1/8	1 1/4	1 1/2	1 3/4	2	2 1/4	2 1/2	2 3/4	3	
1	11 1/2	12	12 1/2										1
1 1/4	12 1/2	12 7/8	13 3/8	13 7/8	14 1/4								1 1/4
1 1/2	13 3/8	13 7/8	14 1/4	14 3/4	15 1/4	16 1/8							1 1/2
1 3/4	14 1/4	14 3/4	15 1/4	15 5/8	16 1/8	17	18						1 3/4
2	15 1/4	15 5/8	16 1/8	16 5/8	17	18	18 7/8	19 3/4					2
2 1/2	17	17 1/2	18	18 3/8	18 7/8	19 3/4	20 3/4	21 5/8	22 5/8	23 1/2			2 1/2
3	18 7/8	19 3/8	19 3/4	20 1/4	20 3/4	21 5/8	22 5/8	23 1/2	24 3/8	25 5/8	26 1/4	27 1/4	3
3 1/2	20 3/4	21 1/8	21 3/8	22 1/8	22 5/8	23 1/2	24 3/8	25 3/8	26 1/4	27 1/4	28 1/8	29	3 1/2
4	22 3/8	23	23 1/2	24	24 3/8	25 3/8	26 1/4	27 1/4	28 1/8	29	30	31	4
4 1/2	24 3/8	24 7/8	25 3/8	25 7/8	26 1/4	27 1/4	28 1/8	29	30	31	31 7/8	32 3/4	4 1/2
5	26 1/4	26 3/4	27 1/4	27 5/8	28 1/8	29	30	31	31 7/8	32 3/4	33 5/8	34 5/8	5
5 1/2	28 1/8	28 5/8	29	29 1/2	30	31	31 7/8	32 3/4	33 5/8	34 5/8	35 1/2	36 1/2	5 1/2
6	30	30 1/2	31	31 3/8	31 7/8	32 3/4	33 5/8	34 5/8	35 1/2	36 1/2	37 3/8	38 3/8	6

NOTE.—Maximum shipping length should not exceed 35 feet.

reason that welded steel is not ordinarily considered reliable. The American Bridge Company's standard loop-bars are shown in Table XX. Loop-bars are made with both single and double loops. Clevises are to be preferred to double loops. Loop-bars bent in the weld should not be used.

Standard Upsets.—Bars upon which screw ends are to be cut, are first upset so that the area through the base of the screw will be in excess of the main body of the bar by a required amount, varying from 16 to 40 per cent. The American Bridge Company's standard upsets for round and square bars are given in Table XXI.

Clevises.—Where small round or square steel bars are used, the ends should be upset and the connection to the pin should be made by

TABLE XXI.

UPSETS FOR ROUND AND SQUARE BARS.

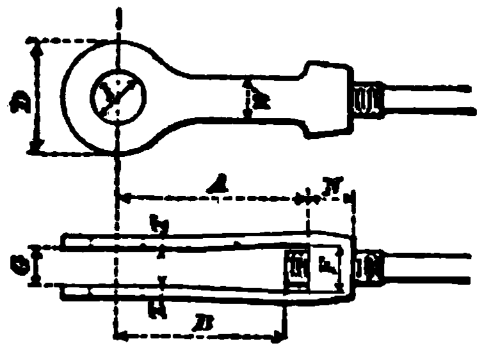
ROUND ○ BARS							SQUARE □ BARS						
ROUND		UPSET					UPSET					SQUARE	
DIAM.	AREA	DIAM.	LENGTH	ADD	AREA AT ROOT	EXCESS AREA	EXCESS AREA	AREA AT ROOT	ADD	LENGTH	DIAM.	AREA	DIAM.
INCHES	SQ. INS.	INCHES	INCHES	INCHES	SQ. INS.	%	%	SQ. INS.	INCHES	INCHES	INCHES	SQ. INS.	INCHES
5/8	0.307	7/8	4	4 1/2	0.420	36.8							5/8
3/4	0.442	1	4	3 7/8	0.550	24.4	20.6	0.694	3 1/2	4	1 1/8	0.563	3/4
7/8	0.601	1 1/4	4	5	0.891	48.3	16.3	0.891	4	4	1 1/4	0.766	7/8
1	0.785	1 3/8	4	4 5/8	1.057	34.7	29.5	1.295	4	4	1 1/2	1.000	1
1 1/8	0.994	1 1/2	4	3 7/8	1.295	30.3	19.7	1.515	4 1/2	4 1/2	1 5/8	1.266	1 1/8
1 1/4	1.227	1 5/8	4 1/2	3 7/8	1.515	23.5	31.1	2.049	4 1/2	4 1/2	1 7/8	1.563	1 1/4
1 3/8	1.485	1 3/4	4 1/2	3 1/2	1.744	17.4	21.7	2.302	4 1/8	5	2	1.891	1 3/8
1 1/2	1.767	2	5	4 3/8	2.302	30.3	34.0	3.023	4 3/4	5	2 1/4	2.250	1 1/2
1 5/8	2.074	2 1/8	5	4 1/4	2.651	27.8	29.6	3.410	4 5/8	5 1/2	2 3/8	2.641	1 5/8
1 3/4	2.405	2 1/4	5	4	3.023	25.7	21.3	3.716	4 1/4	5 1/2	2 1/2	3.063	1 3/4
1 7/8	2.761	2 3/8	5 1/2	4 1/2	3.410	23.9	31.4	4.619	5 1/2	6	2 3/4	3.516	1 7/8
2	3.142	2 1/2	5 1/2	3 7/8	3.716	18.3	27.7	5.107	4 3/4	6	2 7/8	4.000	2
2 1/8	3.547	2 5/8	5 1/2	3 5/8	4.155	17.1	20.2	5.430	4 3/8	6	3	4.516	2 1/8
2 1/4	3.976	2 7/8	6	4 3/8	5.107	28.5	28.6	6.510	5 1/8	6 1/2	3 1/4	5.063	2 1/4
2 3/8	4.430	3	6	4 3/8	5.430	22.6	33.8	7.548	6 1/8	7	3 1/2	5.641	2 3/8
2 1/2	4.909	3 1/8	6 1/2	4 3/8	5.957	21.3	30.7	8.170	6 1/4	8	3 5/8	6.250	2 1/2
2 5/8	5.412	3 1/4	6 1/2	4 1/4	6.510	20.3	35.0	9.305	6 3/4	8	3 7/8	6.891	2 5/8
2 3/4	5.940	3 3/8	7	4 1/4	7.088	19.3	32.1	9.994	6	8	4	7.563	2 3/4
2 7/8	6.492	3 5/8	8	5 1/2	8.170	25.9	37.0	11.329	8	9	4 1/4	8.266	2 7/8
3	7.069	3 3/4	8	5 1/4	8.641	22.2	41.7	12.753	7 1/4	9	4 1/2	9.000	3
3 1/8	7.670	3 7/8	8	5 1/8	9.305	21.3							3 1/8
3 1/4	8.296	4	8	4 7/8	9.994	20.7							3 1/4
3 1/2	9.621	4 1/4	9	5 1/4	11.329	17.7							3 1/2
3 3/4	11.045	4 1/2	9	4 3/4	12.753	15.5							3 3/4

means of clevises. The American Bridge Company's standard clevises are given in Table XXII.

Turnbuckles and Sleeve Nuts.—Eye- or loop-bars are made adjustable by means of turnbuckles or sleeve nuts. Turnbuckles are more

TABLE XXII.

STANDARD CLEVISES. AMERICAN BRIDGE COMPANY STANDARDS.



G = grip = thickness of plate + $\frac{1}{8}''$. $A - B = 1$ in.

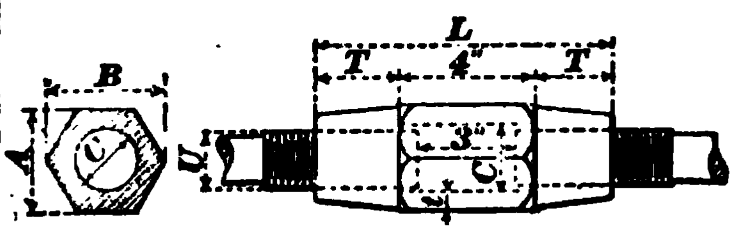
Designation of Clevis. No.	Head. Diameter. D Ins.	Head. Thickness. T Ins.	Diameter of Pin. Max. Ins.	Diameter of Pin. Min. Ins.	W Ins.	F Ins.	A Ins.	Diameter of Upset. Max. Ins.	Diameter of Upset. Min. Ins.	Weight. Lbs.	Designation of Clevis. No.
3	3	½	1½	1	1½	1¼	5	1⅛	1	4	3
4	4	½	2	1¼	1¾	1¾	6	1⅝	1⅛	8	4
5	5	⅝	2½	1½	2¼	2¼	7	2⅛	1½	17	5
6	6	¾	3	2	2¾	2¾	8	2½	2	26	6
7	7	⅞	3½	2½	3¼	3¼	9	2⅞	2¼	40	7

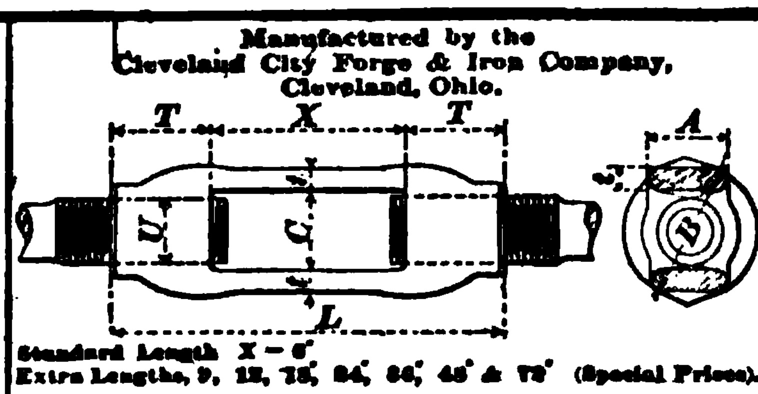
often used than sleeve nuts. The turnbuckle has the advantage that the ends of the bars are visible, while it has the disadvantage that it can be loosened with a bar. The American Bridge Company's standard turnbuckles and sleeve nuts are given in Table XXIII.

Riveted Tension Members.—The difficulty in the design of riveted tension members is the design of the end connections. The rivets in the end connections should be symmetrical with the neutral axis of the member. This is sometimes difficult to attain, and results in large eccentric stresses. In riveted tension members with pin-connections it is usually specified: (1) That the net area through the pin hole must exceed the required net area of the member by 25 per cent, and (2) the area back of the pin hole on a plane through the center of the pin hole and parallel to the axis of the member must be not less than 75 per cent of the area through the pin hole. The net area of the member must be used in calculating the strength of a riveted tension member. In deducting for rivet holes in tension members it is often specified that rupture will be considered equally probable on a trans-

TABLE XXIII.

SLEEVE NUTS AND TURNBUCKLES. AMERICAN BRIDGE COMPANY STANDARDS.
All Dimensions in Inches.





Diam. of Screw U	Length of Thread T	Length of Nut L	Short Diam. A	Long Diam. B	Inside Diam. C	Thickness t	Weight in Lbs.	Weight in Lbs.	Standard Dimensions						Diam. of Screw U
									t	A	B	C	L	T	
7/8	1 1/8	7	1 3/8	1 5/8	1 1/8	1/4	2 3/4	2 1/2	3/8	1 1/4	2 1/4	1 1/4	8 5/8	1 9/16	7/8
1	1 1/2	7	1 5/8	1 7/8	1 1/8	1/4	3	3 1/2	7/16	"	2 7/16	1 5/16	9	1 1/8	1
1 1/8	1 3/4	7 1/2	2	2 5/16	1 3/8	5/16	3 1/2	4	1/2	"	2 9/16	1 7/16	9 3/8	1 11/16	1 1/8
1 1/4	"	"	"	"	"	"	4	5 1/2	"	1 3/8	2 3/4	1 9/16	9 3/4	1 7/8	1 1/4
1 3/8	2	8	2 3/8	2 3/4	1 5/8	3/8	4 1/2	6	"	1 5/8	3 1/16	1 11/16	10 1/8	2 1/16	1 3/8
1 1/2	"	"	"	"	"	"	6 1/2	7	5/8	"	3 3/16	1 3/4	10 1/2	2 1/4	1 1/2
1 5/8	2 1/4	8 1/2	2 3/4	3 3/16	1 7/8	7/16	8	8 1/2	"	1 3/4	3 1/2	2	10 7/8	2 7/16	1 5/8
1 3/4	"	"	"	"	"	"	8 1/2	10	"	2	3 3/4	2 1/8	11 1/4	2 5/8	1 3/4
1 7/8	2 1/2	9	3 1/8	3 5/8	2 1/8	1/2	10	11 1/2	11/16	"	3 7/8	2 5/16	11 5/8	2 13/16	1 7/8
2	"	"	"	"	"	"	11	13	"	2 1/4	4 1/4	2 3/8	12	3	2
2 1/8	2 5/8	9 1/2	3 1/2	4 1/16	2 3/8	9/16	14	15	23/32	2 1/2	4 1/2	2 1/2	12 5/8	3 5/16	2 1/8
2 1/4	"	"	"	"	"	"	15	18	13/16	"	4 3/4	2 11/16	12 3/4	3 3/8	2 1/4
2 3/8	3	10	3 7/8	4 1/2	2 5/8	5/8	18	20	"	2 3/4	4 7/8	2 3/4	13 1/2	3 9/16	2 3/8
2 1/2	"	"	"	"	"	"	19	24	27/32	3	5 3/8	3 1/16	13 1/8	3 3/4	2 1/2
2 5/8	3 1/4	10 1/2	4 1/4	4 15/16	2 7/8	11/16	22	28	15/16	"	5 9/16	3 1/8	13 7/8	3 15/16	2 5/8
2 3/4	"	"	"	"	"	"	23	30	"	3 1/4	5 3/4	3 1/4	14 1/4	4 1/8	2 3/4
2 7/8	3 1/2	11	4 5/8	5 3/8	3 1/8	3/4	27	34	1 1/32	"	6 1/16	3 7/16	14 5/8	4 5/16	2 7/8
3	"	"	"	"	"	"	28	38	"	3 1/2	6 3/8	3 5/8	15	4 1/2	3
3 1/8	3 3/4	11 1/2	5	5 13/16	3 3/8	13/16	34								3 1/8
3 1/4	"	"	"	"	"	"	35	50	1 1/16	4	6 3/4	3 7/8	15 3/4	4 7/8	3 1/4
3 3/8	4	12	5 3/8	6 1/8	3 5/8	7/8	39								3 3/8
3 1/2	"	"	"	"	"	"	40	65	1 7/32	4	7 1/4	4 1/4	16 1/2	5 1/4	3 1/2
3 5/8	4 1/4	12 1/2	5 3/4	6 11/16	3 7/8	15/16	45								3 5/8
3 3/4	"	"	"	"	"	"	47		1 9/16	5	8 1/4	4 7/16	18	6	3 3/4
3 7/8	4 1/2	13	6 1/8	7 1/8	4 1/8	1	52								3 7/8
4	"	"	"	"	"	"	55		1 7/16	5	8 3/4	4 5/8	18	6	4
4 1/4	4 3/4	13 1/2	6 1/2	7 5/16	4 3/8	1 1/16	65								4 1/4
4 1/2	5	14	6 7/8	8	4 3/4	1 1/16	75								4 1/2

verse or diagonal section unless the diagonal section has a net area 30 per cent in excess of the transverse section.

The net area of a tension member, A, required to carry a direct tension, T, with a safe unit stress, f, is $A = T/f$. For methods of calculating the stresses in tension members due to direct and cross-bending forces, see Chapter VIII.

For the calculation of the stresses in an eccentric riveted connection, see Chapter VIII.

The areas to be deducted for rivet holes in tension members are given in Table XXXII.

COMPRESSION MEMBERS.—Some of the common forms of compression members are shown in Fig. 178. The section in (*b*) consisting of two channels and a top cover plate with lacing on the bottom,

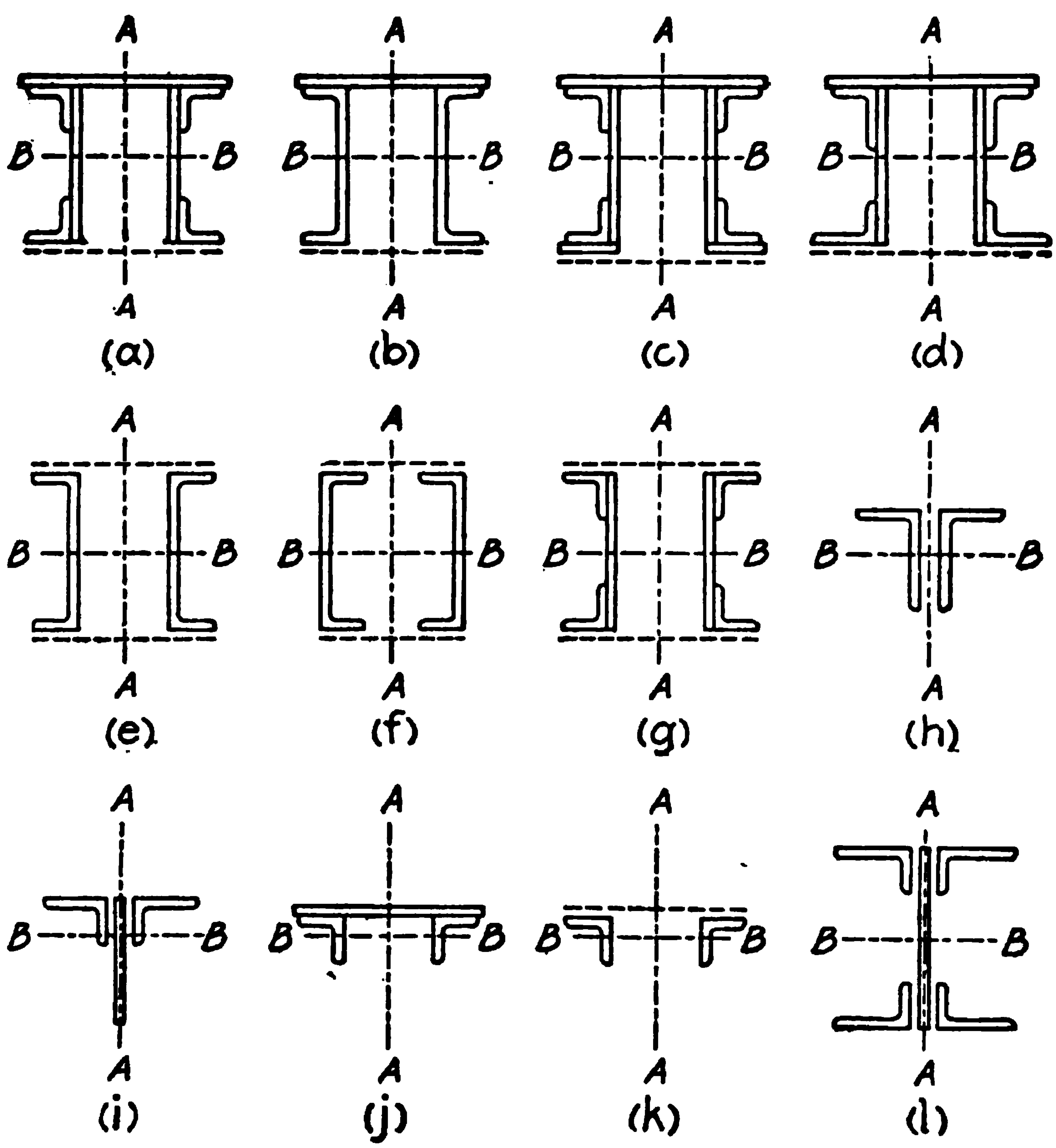


FIG. 178. RIVETED SECTIONS FOR COMPRESSION MEMBERS.

and section (e) consisting of two channels laced on both top and bottom are commonly used for the top chords of high truss pin-connected highway bridges. Sections (a), (c), (d) and (g) are commonly used for long span highway and railway pin-connected bridges. Sections (e), (f), (g) and (j) are used for intermediate posts, while sections (h) to (k) are used for the chords of riveted highway bridges. A type of chord should be selected that will give the desired results and will at the same time give a low cost for fabrication. Where chords are made without a top cover plate, the lacing must be designed to carry the diagonal shear in addition to the usual stresses. The least radius of gyration of sections (a) to (d), inclusive, is approximately four-tenths of the width of the member, while the least radius of gyration of sections (e) to (g), inclusive, is approximately three-eighths of the depth.

In selecting a chord section the radius of gyration should be kept as large as possible, the member at the same time satisfying the following requirements: (1) The thickness of the top cover plate should not be less than 1/40 the distance between the centers of the rivets connecting the plate to the angles or channels. (2) The thickness of the side plates should not be less than 1/30 the distance between the centers of the rivets connecting it to the angles. (3) The angles should not be thinner than three-fourths the thickness of the thickest plate attached to them. (4) The radius of gyration of the member about both axes should be approximately the same.

Design of Compression Members.—The allowable stresses in compression members are given in the standard specifications in this chapter. For the details of the calculations of the moments of inertia, radii of gyration and allowable stresses in compression members, see Chapter VIII, and Chapter XXII.

Lacing.—Lacing bars are used to join the parts of the member together and make it act as a solid member to resist the shear due to bending and the diagonal shear in the member. Lacing bars are commonly made with a thickness of not less than 1/40 the distance between end rivets for single lacing, or 1/60 of the distance between rivets for

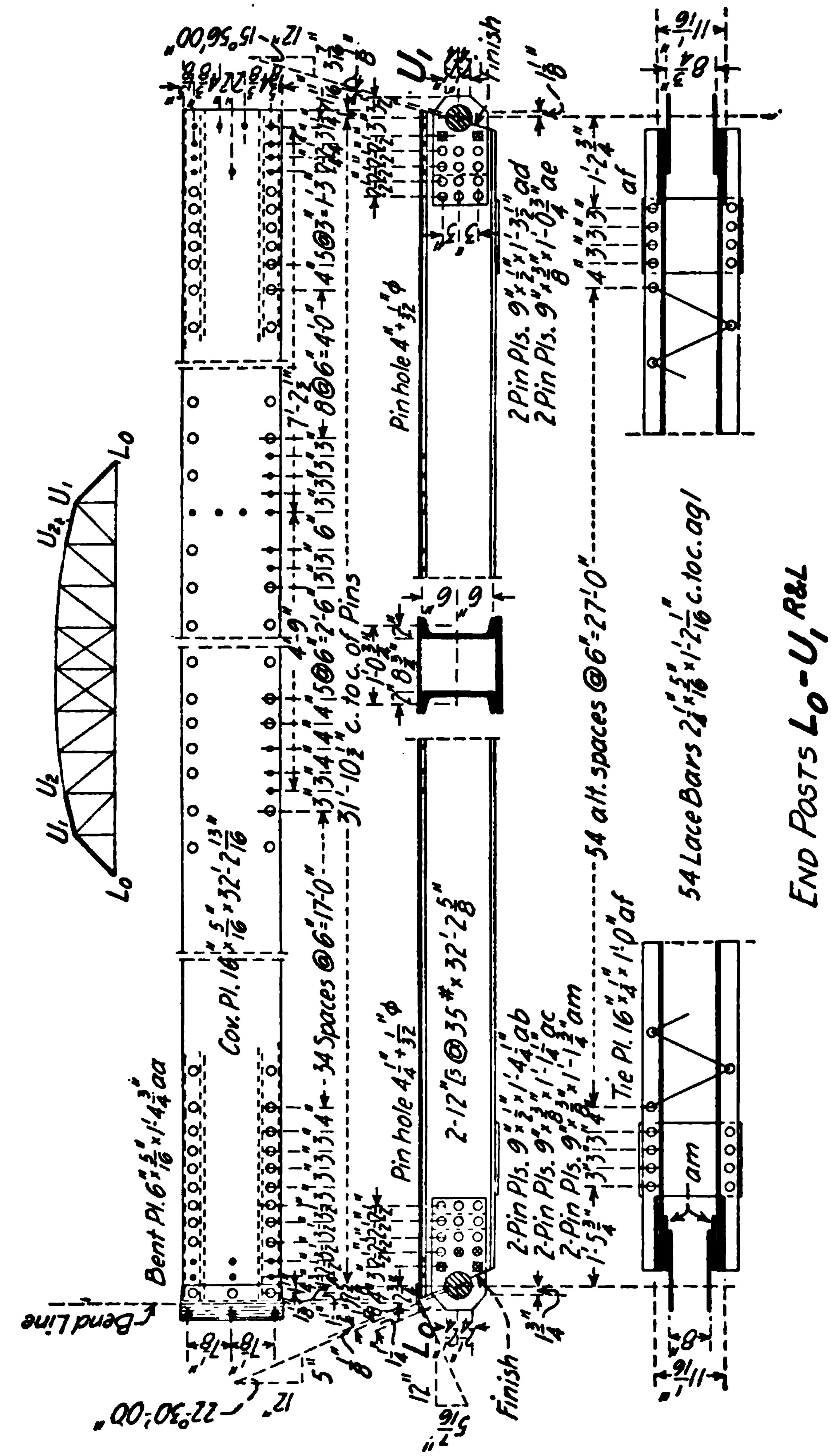


FIG. 179a. DETAIL SHOP PLAN OF END-POST.

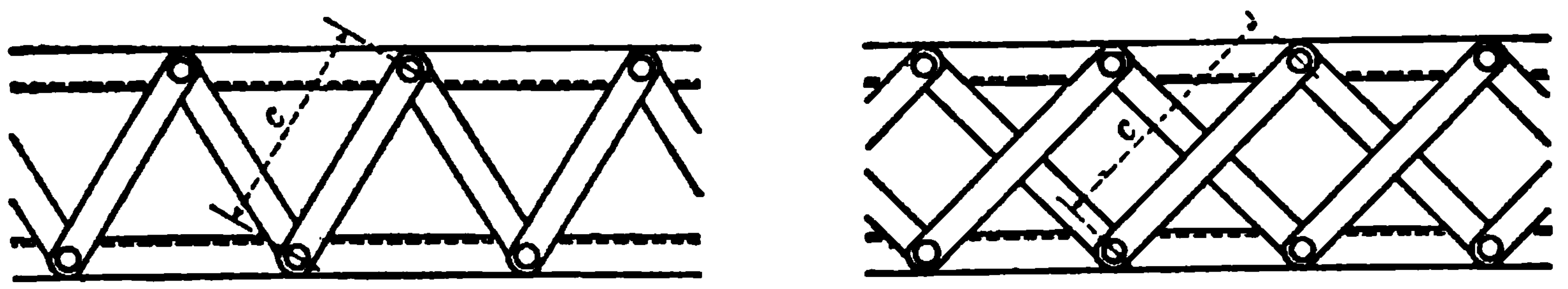
double lacing riveted at the middle. The spacing should be such that the part of the column between the rivets is stronger than the column as a whole. Specifications require that the lacing bars make an angle with the axis of the member of from 60 to 45 degrees. The American Bridge Company's standard lacing bars are given in Table XXIV.

Fig. 179b. Detail Shop Plan of Top Chord.

Design of Lacing Bars.—The lacing bars in a column hold the parts of the column in line, carry part of the diagonal shear, and transfer part of the stress in columns with an eccentric loading. The stresses in the bars required to hold the parts of the column in line are small for stresses in the column within the elastic limit of the material. The maximum diagonal shear, S, in a solid member is $S = \frac{1}{2}P$, where $P =$

TABLE XXIV.

American Bridge Company Standard Lacing Bars.

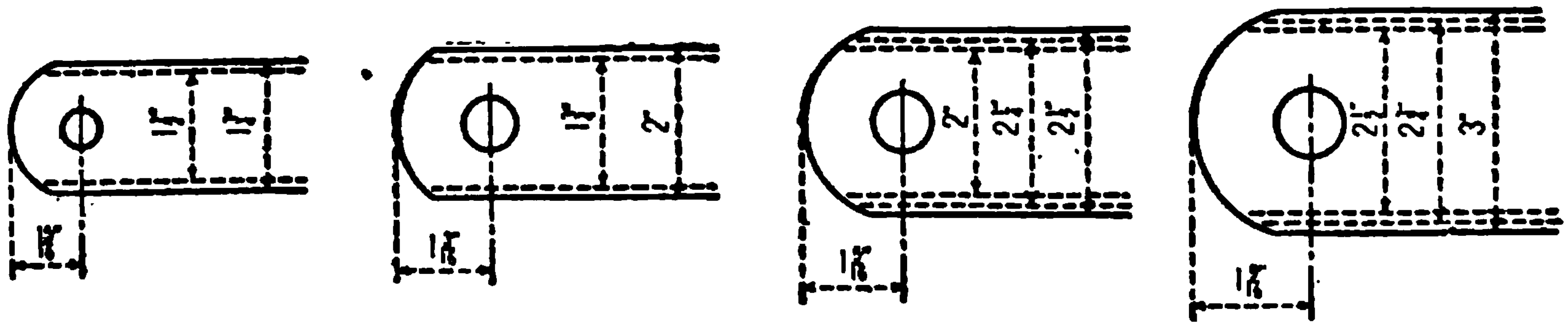


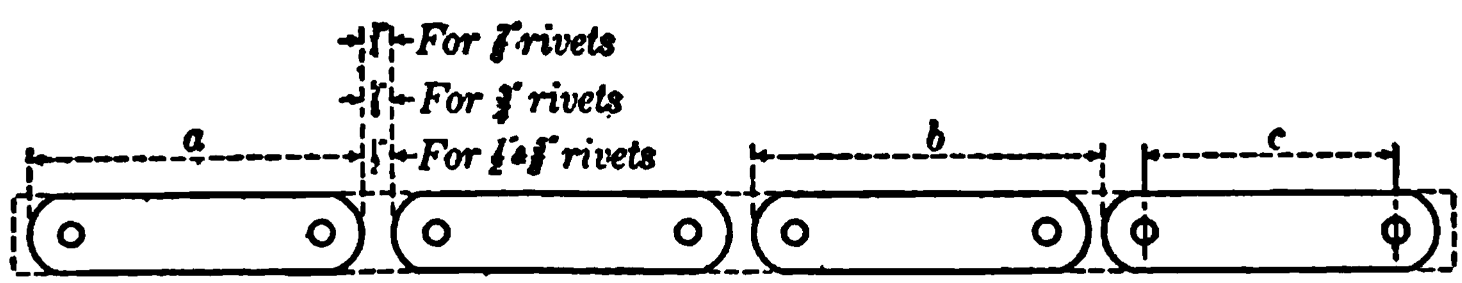
MAXIMUM DISTANCE c IN FEET AND INCHES

FOR GIVEN THICKNESS t OF LACE BAR

THICKNESS OF BAR t INCHES	SINGLE LACING		DOUBLE LACING		THICKNESS OF BAR t INCHES
	$t=\frac{c}{40}$	$t=\frac{c}{50}$	$t=\frac{c}{60}$	$t=\frac{c}{75}$	
⅝	2- 1	2-7¼	3- 1½	3-10⅞	⅝
9/16	1-10½	2-4	2- 9¾	3- 6¼	9/16
½	1- 8	2-1	2- 6	3- 1½	½
7/16	1- 5½	1-9⅞	2- 2¼	2- 8¾	7/16
⅜	1- 3	1-6¾	1-10½	2- 4	⅜
5/16	1- 0½	1-3⅝	1- 6¾	1-11½	5/16
¼	0-10	1-0½	1- 3	1- 6¾	¼

½″ rivet **⅝″ rivet** **¾″ rivet** **⅞″ rivet**





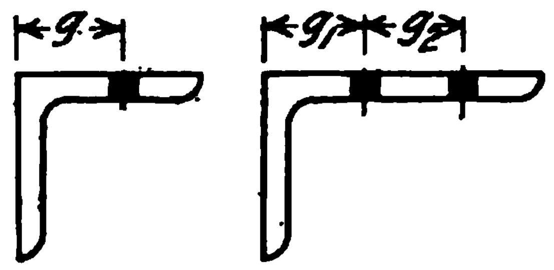
DISTANCE IN INCHES TO BE ADDED TO LENGTH c

WIDTH OF BAR INCHES	FOR FINISHED LENGTH a — DIAMETER OF RIVET ½″	⅝″	¾″	⅞″	FOR ORDERED LENGTH b — DIAMETER OF RIVET ½″	⅝″	¾″	⅞″	WIDTH OF BAR INCHES
3				3⅛				3⅞	3
2¾				3⅛				3⅞	2¾
2½			2⅝	3⅛			3¼	3⅞	2½
2¼			2⅝				3¼		2¼
2		2⅛	2⅝			2⅞	3¼		2
1¾	1⅞	2⅛			2⅜	2⅞			1¾
1½	1⅞				2⅜				1½

the total direct axial load on the member. In columns composed of channels, angles, etc., the flange area is ordinarily sufficient to carry this shear without producing large stresses in the lacing bars. The moment, M', due to the eccentric loading is $M' = P \cdot e$, where $P =$ the total direct load on the column and $e =$ the eccentricity of the loading. The lacing bars will take the shear due to this bending moment, if the flanges are light. It will be seen from the foregoing that the stresses in lacing bars depend (1) upon the make-up of the column, (2) upon the care used in building the column, and (3) upon the eccentricity of the loading.

For a column with a concentric loading, experiments show that the allowable unit stress may be represented by the straight line formula $p = 16{,}000 - 70\, l/r$ lbs. per sq. in., where $p =$ allowable unit stress in the member; $l =$ length of the member, c to c of end connections, and $r =$ radius of gyration of the column, both in inches. Now the allowable unit stress on a short block is 16,000 lbs., and the $70\ l/r$ represents the increase in the fiber stress in the column. Now if we assume that this fiber stress is caused by a horizontal load, W, applied at the center of the height of the column, then $W \cdot l/4 = 70\, I \cdot l/r \cdot y$, where $I =$ moment of inertia of the cross-section of the column $= A \cdot r^2$, where A $=$ the area of the cross-section of the column, and $y =$ the distance from the neutral axis of column to the extreme fiber in the plane parallel to the plane of the lacing bars. Then $W \cdot l/4 = 70 A \cdot r^2 \cdot l/r \cdot y$, and $W = 280\, A \cdot r/y$. Now the shear in the column will be $W/2$, and the shear is $S = 140 A \cdot r/y$, and the stress in a lacing bar will be $= 140\, A \cdot r/y$ $\csc \theta$, where $\theta =$ the angle made by the bar with the axis of the column. This shows that the stresses in the lacing bars in the column with a concentric loading depend upon the make-up of the column, and are independent of the length of the column.

Details of Compression Members.—The details of the end-post L_0U_1 and top chord U_1U_2 of a highway camel-back truss are given in Fig. 179. "Batten plates should be placed as near the ends of the member as practical, should have a thickness not less than $1/40$ the distance between centers of rivets at right angles to the axis of the member, and should have a length not less than the greatest width of

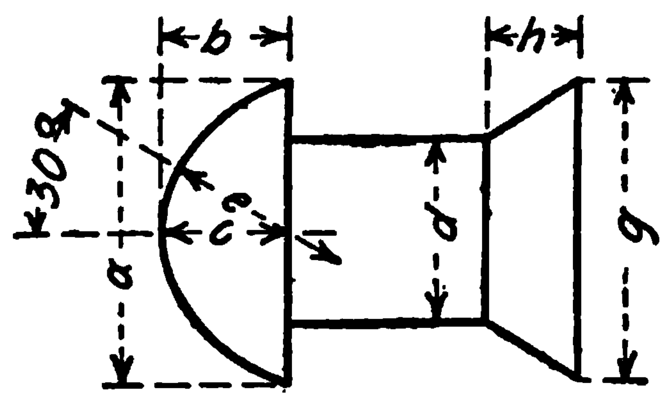


GAGES
in inches

Leg	Gage g	Max. Rivet	Leg	Gage g_1	Gage g_2	Max. Rivet
8	$4\frac{1}{2}$	$\frac{7}{8}$	8	3	3	$\frac{7}{8}$
7	4	$\frac{7}{8}$	7	$2\frac{1}{2}$	3	$\frac{7}{8}$
6	$3\frac{1}{2}$	$\frac{7}{8}$	6	$2\frac{1}{4}$	$2\frac{1}{4}$	$\frac{7}{8}$
5	3	$\frac{7}{8}$	5	2	$1\frac{3}{4}$	$\frac{7}{8}$
4	$2\frac{1}{2}$	$\frac{7}{8}$	When 6" L exceeds $\frac{3}{4}$"			
$3\frac{1}{2}$	2	$\frac{7}{8}$	6	$2\frac{1}{2}$	2	$\frac{7}{8}$
3	$1\frac{3}{4}$	$\frac{7}{8}$				
$2\frac{3}{4}$	$1\frac{5}{8}$	$\frac{3}{4}$				
$2\frac{1}{2}$	$1\frac{3}{8}$	$\frac{3}{4}$				
$2\frac{1}{4}$	$1\frac{1}{4}$	$\frac{5}{8}$				
2	$1\frac{1}{8}$	$\frac{5}{8}$				
$1\frac{3}{4}$	1	$\frac{1}{2}$				
$1\frac{1}{2}$	$\frac{7}{8}$	$\frac{3}{8}$				
$1\frac{3}{8}$	$\frac{7}{8}$	$\frac{3}{8}$				
$1\frac{1}{4}$	$\frac{3}{4}$	$\frac{3}{8}$				
1	$\frac{5}{8}$	$\frac{1}{4}$				

MINIMUM RIVET SPACING

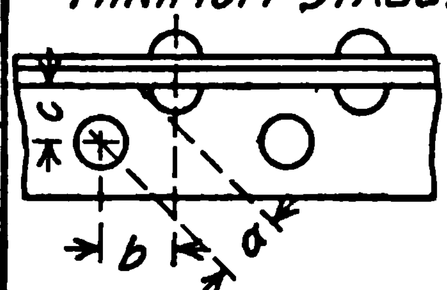
Size of Rivet inches	Min. Distance inches
1	3
$\frac{7}{8}$	$2\frac{5}{8}$
$\frac{3}{4}$	$2\frac{1}{4}$
$\frac{5}{8}$	2
$\frac{1}{2}$	$1\frac{3}{4}$
$\frac{3}{8}$	$1\frac{1}{4}$
$\frac{1}{4}$	1



PROPORTIONS OF RIVETS
in inches

Diameter of Shank d	Full Head Diameter a	Full Head Height b	Radii c	Radii e	Countersunk Diameter g	Countersunk Depth h
1	$1\frac{5}{8}$	$\frac{11}{16}$	$\frac{11}{16}$	$1\frac{1}{32}$	$1\frac{9}{16}$	$\frac{1}{2}$
$\frac{7}{8}$	$1\frac{7}{16}$	$\frac{39}{64}$	$\frac{39}{64}$	$\frac{59}{64}$	$1\frac{3}{8}$	$\frac{7}{16}$
$\frac{3}{4}$	$1\frac{1}{4}$	$\frac{17}{32}$	$\frac{17}{32}$	$\frac{51}{64}$	$1\frac{3}{16}$	$\frac{3}{8}$
$\frac{5}{8}$	$1\frac{1}{16}$	$\frac{29}{64}$	$\frac{29}{64}$	$\frac{43}{64}$	1	$\frac{5}{16}$
$\frac{1}{2}$	$\frac{7}{8}$	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{9}{16}$	$\frac{25}{32}$	$\frac{1}{4}$
$\frac{3}{8}$	$\frac{11}{16}$	$\frac{19}{64}$	$\frac{19}{64}$	$\frac{7}{16}$	$\frac{19}{32}$	$\frac{3}{16}$

MINIMUM STAGGER FOR RIVETS

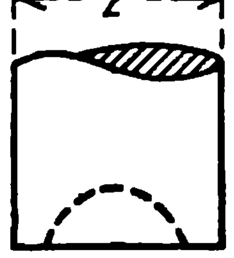


c inches	b in inches, For $\frac{3}{4}$" Rivet $a=1\frac{1}{8}$"	b in inches, For $\frac{7}{8}$" Rivet $a=1\frac{1}{4}$"
$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{1}{2}$
$1\frac{3}{16}$	$1\frac{3}{16}$	$1\frac{7}{16}$
$1\frac{1}{4}$	$1\frac{1}{8}$	$1\frac{3}{8}$
$1\frac{5}{16}$	$1\frac{1}{16}$	$1\frac{5}{16}$
$1\frac{3}{8}$	$\frac{15}{16}$	$1\frac{1}{4}$
$1\frac{7}{16}$	$\frac{7}{8}$	$1\frac{3}{16}$
$1\frac{1}{2}$	$\frac{3}{4}$	$1\frac{1}{8}$
$1\frac{9}{16}$	$\frac{9}{16}$	1
$1\frac{5}{8}$	$\frac{3}{8}$	$\frac{15}{16}$
$1\frac{11}{16}$	0	$\frac{13}{16}$
$1\frac{3}{4}$		$\frac{5}{8}$
$1\frac{13}{16}$		$\frac{7}{16}$
$1\frac{7}{8}$		0

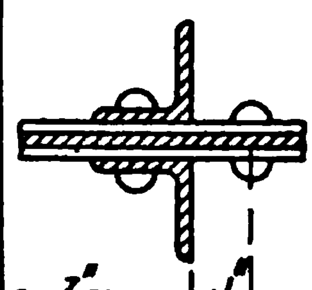
MINIMUM BUTTON SETS

For $\frac{3}{4}$" and $\frac{7}{8}$" Rivets $2\frac{1}{4}$"

For Rivets less than $\frac{3}{4}$" 2"



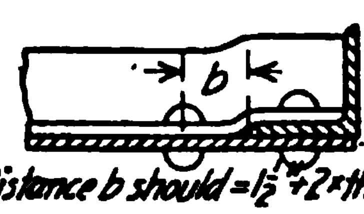
STANDARD CLEARANCE FOR RIVETING



For $\frac{7}{8}$" Rivets $1\frac{1}{4}$"

For $\frac{3}{4}$" Rivets $1\frac{1}{8}$"

RIVETS IN CRIMPED Ls



Distance b should $=1\frac{1}{2}$" $+ 2 \times$ thickness chord Ls, but never less than 2"

FIG. 180. AMERICAN BRIDGE COMPANY'S STANDARDS FOR RIVETS AND RIVETING.

the member or $1\frac{1}{2}$ times the least width of the member." The distance between rivets is $1'\ \frac{3}{4}''$ and the thickness should be greater than $\frac{1}{4}''$ (should be $\frac{5}{16}''$), the length should not be less than $1\frac{1}{2}$ times $12''$ or $1'\ 6''$. The top cover plate satisfies the specifications for minimum

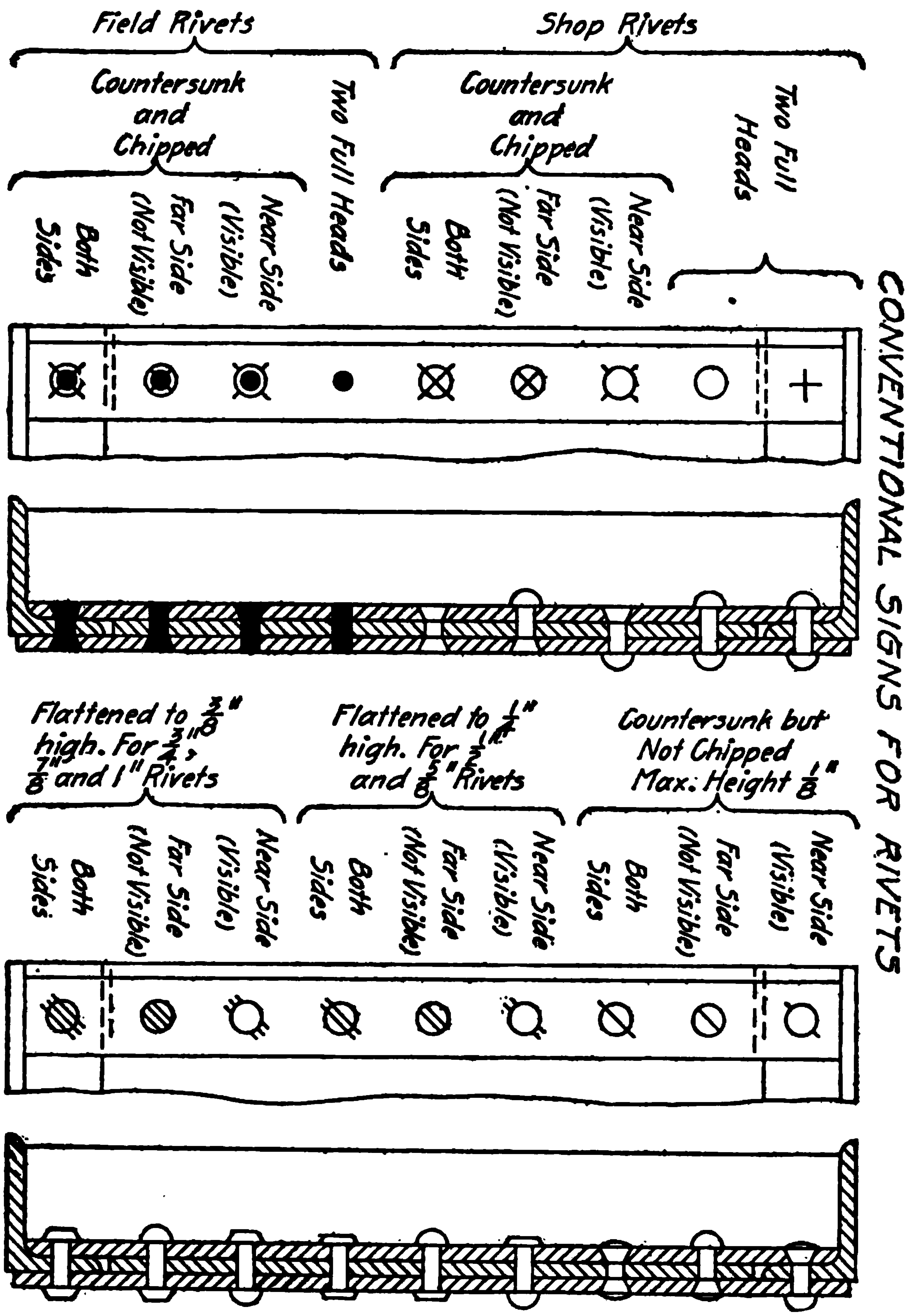


FIG. 181. CONVENTIONAL SIGNS FOR RIVETS.

thickness. The rivet spacing in the line of stress should not be greater than 16 times the thickness of the thinnest outside plate ($16 \times \frac{5}{16}''$ $= 5''$) or 6″. This specification is not fulfilled near the center.

"The rivet spacing should not be less than three diameters of rivet." Rivets are $\frac{3}{4}$ inch and the specifications are fulfilled. "For a length from the end equal to twice the width of the member the rivet spacing should not exceed 4 times the diameter of the rivet." This specification is fulfilled for L_0U_1 and is practically fulfilled for U_1U_2. "Where pin plates are used at least one pin plate must extend 6 inches beyond the edge of the nearest batten plate." This specification is fulfilled for U_1U_2, but is not fulfilled for L_0U_1. The lacing bars satisfy the American Bridge Company's standards for $t = c/50$, but not for $t = c/40$. The rivets in the pin plates are arranged symmetrically with reference to the pin centers. This may or may not be symmetrical with the neutral axis of the member. The details of the joint at pin U_1 are clearly shown. For specifications for riveting see Appendix I.

Angles.—The areas of angles are given in Table XXV, while the weights of angles are given in Table XXVI.

Rivets.—The standard form of rivets, as used by the American Bridge Company for structural and bridge work, are given in Fig. 180, together with other standards for riveting. The spacing of rivets in the legs of angles and the maximum sizes of rivets are given in Fig. 180. The American Bridge Company's conventional signs for rivets are given in Fig. 181. The spacing of rivets in the flanges of channels and the maximum sizes of rivets are given in Table XXVII, and for I beams in Tables XXVIII and XXIX.

The shearing and bearing values of rivets for several different unit stresses are given in Tables XXX and XXXI.

For the allowable stresses on $\frac{5}{8}$ and $\frac{3}{4}$ inch rivets in the trusses, floor system and laterals of highway bridges, according to Cooper's Specifications, see Table LXXVI and Table LXXVII, respectively.

ANCHOR BOLTS.—Three common forms of anchor bolts are given in Fig. 182. The wedge bolt in (a) is driven over the wedge

TABLE XXV.

CARNEGIE ANGLES.

Weights in Pounds Per Lineal Foot.

SIZE	1/8	3/16	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	1 1/16	1 1/8	SIZE
8 × 8							26.4	29.5	32.7	35.8	38.9	42.0	45.0	48.0	51.0	54.0	56.9	8 × 8
6 × 6					14.9	17.2	19.6	21.9	24.2	26.5	28.7	30.9	33.1	35.3	37.4			6 × 6
5 × 5					12.3	14.3	16.2	18.1	20.0	21.8	23.6	25.4	27.2	28.9	30.6			[redacted] × 5
4 × 4				8.2	9.8	11.3	12.8	14.3	15.7	17.1	18.5	19.9						4 × 4
3½ × 3½				7.1	8.5	9.8	11.1	12.3	13.6	14.8	16.0	17.1						3½ × [redacted]
3 × 3			4.9	6.1	7.2	8.3	[redacted]	10.4	11.4									3 × 3
2¾ × 2¾			4.5	5.5	6.6	7.6	8.5											2¾ × 2¾
2½ × 2½		3.1	4.0	5.0	5.9	6.8	7.7											2½ × 2½
2¼ × 2¼		2.8	3.7	4.5	5.3	6.1	6.8											2¼ × 2¼
2 × 2		2.5	3.2	4.0	4.7	5.3												2 × 2
1¾ × 1¾		2.1	2.8	3.4	4.0	4.6												1¾ × 1¾
1½ × 1½	1.2	1.8	2.4	2.9	3.4													1½ × 1½
1¼ × 1¼	1.0	1.5	1.9	2.4														1¼ × 1¼
1 × 1	0.8	1.2	1.5															1 × 1
SIZE	1/8	3/16	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	1 1/16	1 1/8	SIZE
7 × 3½						15.0	17.0	19.0	21.0	23.0	24.9	26.8	28.7	30.5	32.3			7 × 3½
6 × 4					12.3	14.3	16.2	18.1	20.0	21.8	23.6	25.4	27.2	28.9	30.6			6 × 4
6 × 3½					11.7	13.5	15.3	17.1	18.9	20.6	22.3	24.0	25.7	27.3	28.9			6 × 3½
5 × 4					11.0	12.8	14.5	16.2	17.8	19.5	21.1	22.6	24.2					5 × 4
5 × 3½				8.7	10.4	12.0	13.6	15.2	16.8	18.3	19.8	21.3	22.7					5 × 3½
5 × 3				8.2	9.8	11.3	12.8	[redacted]	15.7	17.1	18.5	19.9						5 × 3
4 × 3½				7.7	9.1	10.5	11.9	13.3	14.6	15.9	17.2	18.5						4 × 3½
4 × 3				7.1	8.5	9.8	11.1	12.3	13.6	14.8	16.0	17.1						4 × 3
3½ × 3				6.6	7.8	9.1	10.2	11.4	12.5	13.6	14.7	15.7						3½ × 3
3½ × 2½			4.9	6.1	7.2	8.3	9.4	10.4	11.4	[redacted]								3½ × 2½
3 × 2½			4.5	5.5	6.6	7.6	8.5	9.5										3 × 2½
3 × 2			4.0	5.0	5.9	6.8	7.7											3 × 2
2½ × 2		2.8	3.7	4.5	5.3	6.1	6.8											2½ × 2
SIZE	1/8	3/16	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	1 1/16	1 1/8	SIZE

TABLE XXVI.

Carnegie Angles.

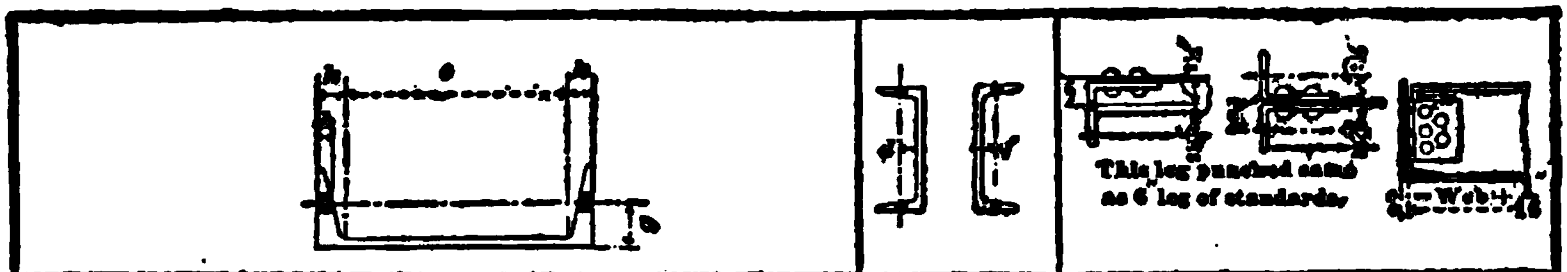
Areas in Square Inches.

Size	1/8	3/16	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	1 1/16	1 1/8	Size
8 x 8							7.75	8.68	9.61	10.53	11.44	12.34	13.23	14.12	15.00	15.87	16.73	8 x 8
6 x 6					4.36	5.06	5.75	6.43	7.11	7.78	8.44	9.09	9.74	10.37	11.00			6 x 6
*5 x 5					3.61	[illegible]	4.75	5.31	5.86	[illegible]	6.94	7.48	7.99	8.50	9.00			*5 x 5
4 x 4				2.40	2.86	[illegible]	3.75	[illegible]	4.61	5.03	5.44	5.84						4 x 4
3½ x 3½				2.09	2.48	2.87	3.25	[illegible]	[illegible]	4.34	4.69	5.03						3½ x 3½
3 x 3			1.44	1.78	2.11	2.43	2.75	3.06	3.36									3 x 3
*2¾ x 2¾			1.31	1.62	1.92	2.22	2.50											*2¾ x 2¾
2½ x 2½		0.90	1.19	1.47	1.73	2.00	2.25											2½ x 2½
*2¼ x 2¼		0.81	1.06	1.31	1.56	1.78	2.00											*2¼ x 2¼
2 x 2		0.72	0.94	1.15	1.36	1.56												2 x 2
1¾ x 1¾		0.62	0.81	1.00	1.17	1.30												1¾ x 1¾
1½ x 1½	0.36	0.53	0.69	0.84	0.99													1½ x 1½
1¼ x 1¼	0.30	0.43	0.56	0.69														1¼ x 1¼
1 x 1	0.24	0.34	0.44															1 x 1
Size	1/8	3/16	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	1 1/16	1 1/8	Size
*7 x 3½						[illegible]	5.00	5.59	6.17	6.75	7.31	7.87	8.42	8.97	9.50			*7 x 3½
6 x 4					3.61	4.18	4.75	5.31	5.86	6.41	6.94	7.47	7.99	8.50	9.00			6 x 4
6 x 3½					3.42	3.97	[illegible]	5.03	5.55	[illegible]	6.56	[illegible]	[illegible]	8.03	8.50			6 x 3½
*5 x 4					[illegible]	3.75	4.28	4.78	5.23	5.72	6.19	6.65	7.11					*5 x 4
5 x 3½				2.56	3.05	3.53	4.00	4.47	4.92	5.37	5.81	6.25	6.67					5 x 3½
5 x 3				2.40	2.86	3.31	3.75	4.18	4.61	[illegible]	5.44	5.84						5 x 3
*4 x 3½				2.25	2.67	3.09	3.50	3.90	4.30	[illegible]	5.06	5.43						*4 x 3½
4 x 3				2.09	2.48	2.87	3.25	3.62	3.98	4.34	4.69	5.03						4 x 3
3½ x 3				1.93	2.30	2.65	3.00	3.34	3.67	4.00	4.31	4.62						3½ x 3
3½ x 2½			1.44	1.78	2.11	2.43	[illegible]	3.06	3.30	3.66								3½ x 2½
3 x 2½			1.31	1.62	1.92	2.22	2.50	2.78										3 x 2½
3 x 2			1.19	1.47	1.73	2.00	2.25											3 x 2
2½ x 2		0.81	1.06	1.31	1.55	1.78	2.00											2½ x 2
Size	1/8	3/16	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	1 1/16	1 1/8	Size

Angles marked * are special.

TABLE XXVII.

CARNEGIE CHANNELS.



]	Weight per Foot	Flange	Web	Gauge g	Tang't. t	Dist. k	Grip b	Max. Rivet or Bolt	Dist. d	Gauge f	Dist. e	Dist. c	Weight per Foot	[
15	55.00	3 13/16	13/16	2¼	12¼	1¼	⅝	¾	1¼	2 7/16	3 3/8	⅞	55.00	15
	50.00	3 23/32	23/32	"	"	"	"		"	2 5/16	3¼	13/16	50.00	
	45.00	3⅝	⅝	2	"	"	"		"	2¼	3⅛	11/16	45.00	
	40.00	3 17/32	17/32	"	"	"	"		"	2 3/16	3	⅝	40.00	
	35.00	3 7/16	7/16	"	"	"	"		"	2⅛	2 15/16	½	35.00	
	33.00	3 13/32	13/32	"	"	"	"		"	2 1/16	2 27/32	7/16	33.00	
12	40.00	3 13/32	¾	"	10	1	½	¾	1 3/16	2 3/16	3¼	13/16	40.00	12
	35.00	3 11/32	⅝	"	"	"	"		"	2⅛	3⅛	11/16	35.00	
	30.00	3 3/16	½	1¾	"	"	"		"	1 15/16	3	9/16	30.00	
	25.00	3 1/16	13/32	"	"	"	"		"	1⅞	2⅞	7/16	25.00	
	20.50	2 15/16	9/32	"	"	"	"		"	1¾	2 13/16	⅜	20.50	
10	35.00	3 3/16	13/16	"	8¼	⅞	"	¾	1⅛	2 1/16	3 3/16	⅞	35.00	10
	30.00	3 1/32	11/16	"	"	"	"		"	1 15/16	3 1/16	¾	30.00	
	25.00	2⅞	17/32	"	"	"	7/16		"	1 13/16	3 1/16	⅝	25.00	
	20.00	2¾	⅜	1½	"	"	"		"	1⅝	2⅞	7/16	20.00	
	15.00	2 19/32	¼	"	"	"	"		"	1½	2¾	5/16	15.00	
9	25.00	2 13/16	⅝	"	7¼	⅞	"	¾	1⅛	1¾	3⅛	11/16	25.00	9
	20.00	2 21/32	7/16	"	"	"	"		"	1 9/16	2 15/16	½	20.00	
	15.00	2½	9/32	1⅜	"	"	"		"	1 13/16	2 13/16	⅜	15.00	
	13.25	2 7/16	7/32	"	"	"	"		"	1 11/16	2¾	5/16	13.25	
8	21.25	2⅝	9/16	1½	6¼	"	⅜	¾	1 1/16	1 13/16	3 1/16	⅝	21.25	8
	18.75	2 17/32	½	"	"	"	"		"	1 11/16	3	9/16	18.75	
	16.25	2 7/16	13/32	"	"	"	"		"	1 9/16	2⅞	½	16.25	
	13.75	2 11/32	5/16	1⅜	"	"	"		"	1½	2 13/16	⅜	13.75	
	11.25	2¼	7/32	"	"	"	"		"	1 7/16	2¾	5/16	11.25	
7	19.75	2½	⅝	1½	5¼	¾	"	⅝	1 1/16	1 11/16	3 1/16	11/16	19.75	7
	17.25	2 13/32	17/32	"	"	"	"		"	1 9/16	3	9/16	17.25	
	14.75	2 5/16	7/16	"	"	"	"		"	1½	2 15/16	½	14.75	
	12.25	2 3/16	5/16	1¼	"	"	"		"	1⅜	2 13/16	⅜	12.25	
	9.75	2 3/32	7/32	"	"	"	"		"	1 5/16	2¾	¼	9.75	
6	15.50	2 9/32	9/16	"	4¼	"	"	⅝	15/16	1⅝	3 1/16	⅝	15.50	6
	13.00	2 5/32	7/16	"	"	"	"		"	1½	2 15/16	½	13.00	
	10.50	2 1/32	5/16	"	"	"	"		"	1⅜	2 13/16	⅜	10.50	
	8.00	1 29/32	3/16	1⅛	"	"	"		"	1¼	2 11/16	¼	8.00	
5	11.50	2 1/32	15/32	"	3¼	⅝	5/16	½	⅞	1 7/16	3	9/16	11.50	5
	9.00	1⅞	11/32	"	"	"	"		"	1 5/16	2 13/16	7/16	9.00	
	6.50	1¾	3/16	"	"	"	"		"	1 3/16	2 11/16	¼	6.50	
4	7.25	1 23/32	5/16	1	2¼	⅝	"	½	¾	1 1/16	2 13/16	⅜	7.25	4
	6.25	1 21/32	¼	"	"	"	"		"	1	2¾	5/16	6.25	
	5.25	1 9/16	3/16	"	"	"	"		"	15/16	2 11/16	¼	5.25	
3	6.00	1 19/32	⅜	⅞	1¼	½	¼	½	⅝	1	2⅞	7/16	6.00	3
	5.00	1½	¼	"	"	"	"		"	⅞	2¾	5/16	5.00	
	4.00	1 13/32	3/16	"	"	"	"		"	13/16	2 11/16	¼	4.00	

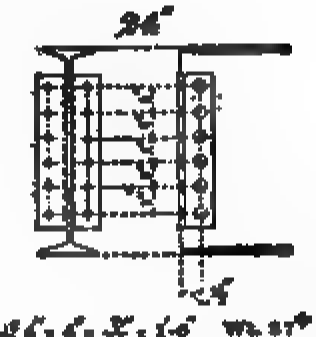
TABLE XXVIII.

Carnegie I-Beams.

I	Weight per foot	Flange	Web	Gauge g	[illegible] t	Dist. k	Grip b	Max. rivet or bolt	[illegible]	Wall Pl.	Dist. a	[illegible] c	Weight per foot	I
Inches	Pounds	Inches	Inches	Inches	Inches	Inches	Inches	Inches	Inches		Inches	Inches	Pounds	Inches
24	100.0	7 1/4	3/4	4	20 3/4	1 5/8	7/8	1	16	16"×1"×1'-4" 73#	5 5/8	1/16	100.0	24
	95.0	7 3/16	11/16	"	"	"	"				5 11/16	1/16	95.0	
	90.0	7 1/8	5/8	"	"	"	"				5 3/4	1/8	90.0	
	85.0	7 1/16	9/16	"	"	"	"				5 13/16	1/8	85.0	
	80.0	7	1/2	"	"	"	"				5 7/8	3/16	80.0	
20	100.0	7 5/16	7/8	"	16 1/2	1 3/4	15/16	7/8	16	16"×1"×1'-4" 73#	5 7/8	1/8	100.0	20
	95.0	7 3/16	13/16	"	"	"	"				5 15/16	1/8	95.0	
	90.0	7 1/8	3/4	"	"	"	"				5 7/8	3/16	90.0	
	85.0	7 1/16	11/16	"	"	"	"				5 15/16	3/16	85.0	
	80.0	7	5/8	"	"	"	"				6	1/4	80.0	
	75.0	6 13/32	21/32	"	17	1 1/2	3/4				5 13/16	3/16	75.0	
	70.0	6 5/16	9/16	"	"	"	"				5 7/8	1/4	70.0	
	65.0	6 1/4	1/2	"	"	"	"				5 7/8	5/16	65.0	
18	70.0	6 1/4	23/32	3 1/2	15 1/4	1 3/8	3/4	7/8	16	15"×1"×1'-4" 73#	5 3/4	7/32	70.0	18
	65.0	6 3/16	5/8	"	"	"	"				5 3/4	1/4	65.0	
	60.0	6 1/16	9/16	"	"	"	"				5 13/16	1/4	60.0	
	55.0	6	15/32	"	"	"	"				5 7/8	5/16	55.0	
15	100.0	6 25/32	1 3/16	"	11	2	1	7/8	12	12"×3/4"×1'-4" 41#	5 15/16	0	100.0	15
	95.0	6 23/32	1 1/8	"	"	"	"				5 15/16	1/16	95.0	
	90.0	6 5/8	1 1/16	"	"	"	"				6	1/16	90.0	
	85.0	6 9/16	1	"	"	"	"				6 1/16	1/8	85.0	
	80.0	6 1/2	15/16	"	"	"	"				6 1/16	1/8	80.0	
	75.0	6 15/32	7/8	"	11 3/4	1 5/8	3/4	3/4			5 7/8	1/8	75.0	
	70.0	6 3/8	25/32	"	"	"	"				5 15/16	3/16	70.0	
	65.0	6 9/32	11/16	"	"	"	"				5 31/32	7/32	65.0	
	60.0	6	25/32	"	"	"	"				5 7/8	1/4	60.0	
	55.0	5 3/4	21/32	3 1/2	12 1/4	1 3/8	5/8				5 7/8	1/4	55.0	
	50.0	5 5/8	17/32	"	"	"	"				5 15/16	1/4	50.0	
	45.0	5 17/32	7/16	"	"	"	"				5 15/16	5/16	45.0	
	42.0	5 1/2	13/32	"	"	"	"				5 7/16	1/4	42.0	
12	55.0	5 5/8	13/16	3 1/4	9 1/2	1 3/8	11/16	3/4	12	12"×1/2"×1'-0" 31#	5 13/16	1/16	55.0	12
	50.0	5 1/2	11/16	"	"	"	"				5 15/16	1/16	50.0	
	45.0	5 3/8	9/16	3	"	"	"				5 13/16	1/8	45.0	
	40.0	5 1/4	7/16	"	"	"	"				5 3/8	3/16	40.0	
	35.0	5 3/32	7/16	"	9 3/4	1 1/8	5/8				5 1/16	3/16	35.0	
	31.5	5	11/32	"	"	"	"				5 3/8	1/4	31.5	

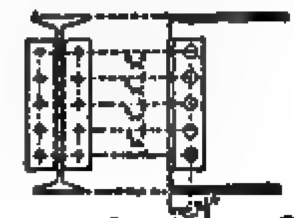
Standard Framing

24"



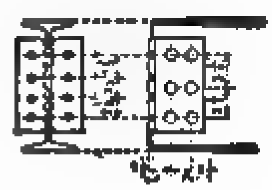
2-L 6"×4"×7/16"×1'-6" Wt. 37#

18" and 20"



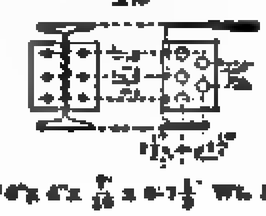
2-L 6"×4"×7/16"×1'-6" Wt. 31#

15"



2-L 6"×4"×3/8"×0'-10" Wt. 27#

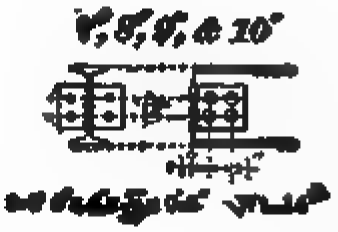
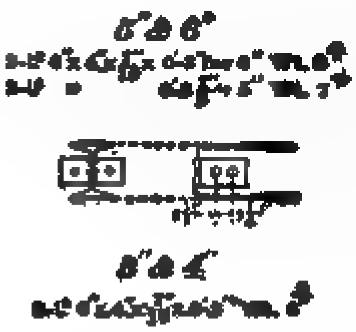
12"



2-L 6"×4"×3/8"×0'-7 1/2" Wt. 20#

TABLE XXIX.

Carnegie I-Beams.

I	Weight per foot	[illegible]	[illegible]	*g*	*s*	*h*	*b*	Max. rivet or bolt	[illegible]	Wall pl.	Standard framing	*a*	*c*	Weight per foot	I
10	[illegible]	[illegible]	[illegible]	3	6	1	[illegible]	[illegible]	6	[illegible]		[illegible]	[illegible]	[illegible]	10
	[illegible]	[illegible]	[illegible]	2¼	"	"	"					[illegible]	[illegible]	[illegible]	
	[illegible]	[illegible]	[illegible]	"	"	"	"					[illegible]	[illegible]	[illegible]	
	[illegible]	[illegible]	[illegible]	"	"	"	"					[illegible]	[illegible]	[illegible]	
9	[illegible]	[illegible]	[illegible]	"	7	"	"	[illegible]	[illegible]			[illegible]	[illegible]	[illegible]	9
	[illegible]	[illegible]	[illegible]	"	"	"	"					[illegible]	[illegible]	[illegible]	
	[illegible]	[illegible]	[illegible]	2¼	"	"	"					[illegible]	[illegible]	[illegible]	
	[illegible]	[illegible]	[illegible]	"	"	"	"					[illegible]	[illegible]	[illegible]	
8	[illegible]	[illegible]	[illegible]	"	[illegible]	[illegible]	"	[illegible]	6	[illegible]		[illegible]	[illegible]	[illegible]	8
	[illegible]	[illegible]	[illegible]	"	"	"	"					[illegible]	[illegible]	[illegible]	
	[illegible]	[illegible]	[illegible]	[illegible]	"	"	"					[illegible]	[illegible]	[illegible]	
	[illegible]	4	[illegible]	"	"	"	"					[illegible]	[illegible]	[illegible]	
7	[illegible]	[illegible]	[illegible]	"	[illegible]	"	[illegible]	[illegible]	6			[illegible]	[illegible]	[illegible]	7
	17.5	[illegible]	[illegible]	"	"	"	"					[illegible]	[illegible]	17.5	
	[illegible]	[illegible]	[illegible]	"	"	"	"					[illegible]	[illegible]	[illegible]	
6	17.25	[illegible]	[illegible]	2	[illegible]	[illegible]	"	[illegible]	6	[illegible]		[illegible]	[illegible]	17.25	6
	14.75	[illegible]	[illegible]	"	"	"	"					[illegible]	[illegible]	14.75	
	12.25	[illegible]	[illegible]	"	"	"	"					[illegible]	[illegible]	12.25	
5	14.75	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	6			[illegible]	[illegible]	14.75	5
	12.25	3¼	[illegible]	"	"	"	"					[illegible]	[illegible]	12.25	
	9.75	3	[illegible]	"	"	"	"					[illegible]	[illegible]	9.75	
4	10.5	[illegible]	[illegible]	[illegible]	[illegible]	"	"	[illegible]	6	[illegible]		[illegible]	[illegible]	10.5	4
	9.5	[illegible]	[illegible]	"	"	"	"					[illegible]	[illegible]	9.5	
	8.5	[illegible]	[illegible]	"	"	"	"					[illegible]	[illegible]	8.5	
	7.5	[illegible]	[illegible]	"	"	"	"					[illegible]	[illegible]	7.5	
3	7.5	[illegible]	[illegible]	1½	[illegible]	"	[illegible]	[illegible]	6			[illegible]	[illegible]	7.5	3
	6.5	[illegible]	[illegible]	"	"	"	"					[illegible]	[illegible]	6.5	
	5.5	[illegible]	[illegible]	"	"	"	"					[illegible]	[illegible]	5.5	

All rivets in standard framing angles are ¾" diam.
Weights of " " " include weight of shop rivets only.
When beams frame opposite each other into another beam with web thickness less than $\frac{5}{16}$" or where beams of short span lengths are loaded to their full capacity, it may be necessary to use framing angles of greater strength than the standards.
See table below for minimum span lengths.

I	Weight	Span in ft.	I	Weight	Span in ft.	I	Weight	Span in ft.	I	Weight	Span in ft.	I	Weight	Span in ft.	I	Weight	Span in ft.	I	Weight	Span in ft.
24	80.0	22.0				15	60.0	20.0							8	18.0	5.5	5	9.75	4.0
20	80.0	22.0	18	55.0	14.0	"	60.0	15.5	12	40.0	11.5	10	25.0	9.0	7	15.0	4.0	4	7.5	3.0
"	65.0	18.0				"	42.0	11.0	"	31.5	9.0	9	21.0	7.0	6	12.25	6.0	3	5.5	2.0

TABLE XXX.

Shearing and Bearing Value of Rivets.

Values Above or to Right of Upper Zigzag Lines are Greater Than Double Shear. Values Below or to Left of Lower Zigzag Lines are Less Than Single Shear.

Diam. of Rivet.		Area in Sq. Ins.	Single Shear at 6,000 Pounds	Bearing Value for Different Thickness of Plate at 12,000 Pounds per Square Inch.												
Frac.	Dec'l.			$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$	1
$\frac{3}{8}$	.375	.1104	660	1,130	1,410	1,690										
$\frac{1}{2}$	.500	.1963	1,180	1,500	1,880	2,250	2,630	3,000								
$\frac{5}{8}$	.625	.3068	1,840	1,880	2,340	2,810	3,280	3,750	4,220	4,690						
$\frac{3}{4}$	.750	.4418	2,650	2,250	2,810	3,380	3,940	4,500	5,160	5,630	6,190	6,750				
$\frac{7}{8}$	.875	.6013	3,610	2,630	3,280	3,940	4,590	5,250	5,910	6,560	7,220	7,880	8,530	9,190	9,840	
1	1.000	.7854	4,710	3,000	3,750	4,500	5,250	6,000	6,750	7,500	8,250	9,000	9,750	10,500	11,250	12,000

Diam. of Rivet.		Area in Sq. Ins.	Single Shear at 7,500 Pounds.	Bearing Value for Different Thickness of Plate at 15,000 Pounds per Square Inch.												
Frac.	Dec'l.			$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$	1
$\frac{3}{8}$	.375	.1104	830	1,410	1,760	2,110										
$\frac{1}{2}$	.500	.1963	1,470	1,880	2,340	2,810	3,280	3,750								
$\frac{5}{8}$	.625	.3068	2,300	2,340	2,930	3,520	4,100	4,690	5,280	5,860						
$\frac{3}{4}$	.750	.4418	3,310	2,810	3,520	4,220	4,920	5,630	6,330	7,030	7,720	8,440				
$\frac{7}{8}$	.875	.6013	4,510	3,280	4,100	4,920	5,740	6,560	7,380	8,200	9,030	9,850	10,670	11,480	12,300	
1	1.000	.7854	5,890	3,750	4,690	5,620	6,560	7,500	8,440	9,380	10,310	11,250	12,190	13,130	14,060	15,000

TABLE XXXI.

SHEARING AND BEARING VALUE OF RIVETS.

Diam. of Rivet.		Area in Sq. Ins.	Single Shear at 11,000 Pounds	Bearing Value for Different Thickness of Plate at 22,000 Pounds per Square Inch.												
Frac.	Dec'l.			1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1
3/8	.375	.1104	1,210	2,060	2,580	3,090										
1/2	.500	.1963	2,160	2,750	3,440	4,130	4,820	5,500								
5/8	.625	.3068	3,370	3,440	4,300	5,160	6,020	6,880	7,740	8,600						
3/4	.750	.4418	4,860	4,130	5,160	6,190	7,220	8,250	9,280	10,320	11,340	12,380				
7/8	.875	.6013	6,610	4,810	6,020	7,220	8,430	9,630	10,840	12,040	13,240	14,440	15,640	16,840	18,050	
1	1.000	.7854	8,640	5,500	6,880	8,250	9,630	11,000	12,380	13,750	15,130	16,500	17,880	19,520	20,630	22,000

Diam. of Rivet.		Area in Sq. Ins.	Single Shear at 12,000 Pounds	Bearing Value for Different Thickness of Plate at 24,000 Pounds per Square Inch.												
Frac.	Dec'l.			1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1
3/8	.375	.1104	1,320	2,250	2,810	3,380										
1/2	.500	.1963	2,360	3,000	3,750	4,500	5,250	6,000								
5/8	.625	.3068	3,680	3,750	4,690	5,620	6,560	7,500	8,440	9,370						
3/4	.750	.4418	5,300	4,500	5,620	6,750	7,870	9,000	10,120	11,250	12,370	13,500				
7/8	.875	.6013	7,220	5,250	6,560	7,870	9,190	10,500	11,810	13,120	14,440	15,750	17,060	18,370		
1	1.000	.7854	9,430	6,000	7,500	9,000	10,500	12,000	13,500	15,000	16,500	18,000	19,500	21,000	22,500	24,000

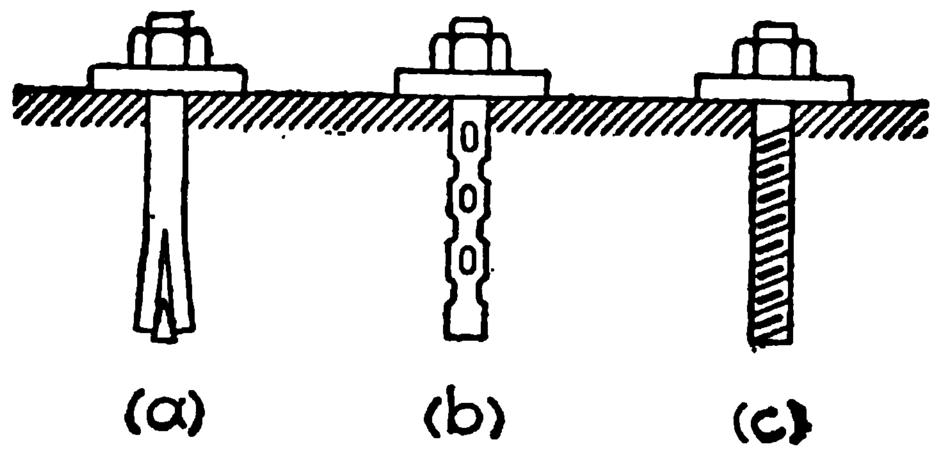


FIG. 182. ANCHOR BOLTS.

TABLE XXXII.

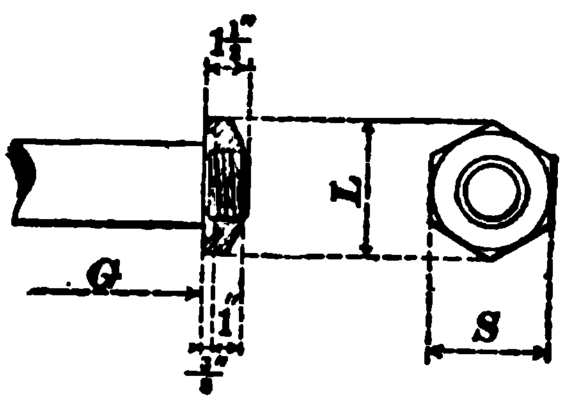
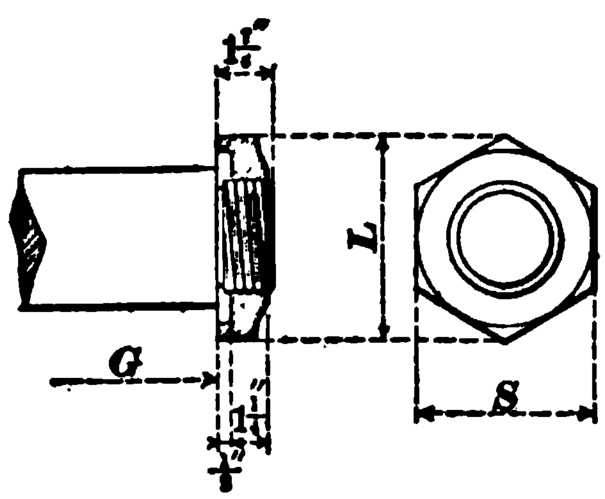
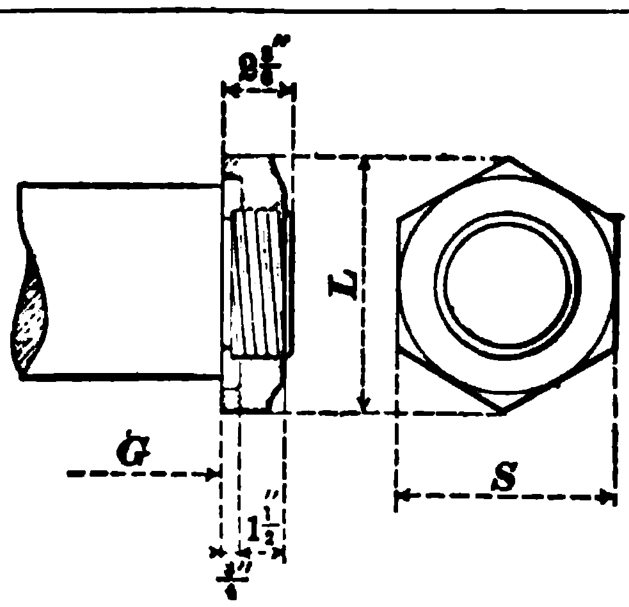
TABLE OF AREAS IN SQUARE INCHES, TO BE DEDUCTED FROM RIVETED PLATES OR SHAPES TO OBTAIN NET AREAS.

Thickness Plates in Inches.	Size of Hole. Inches.													
	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	1 1/16
1/4	.06	.08	.09	.11	.13	.14	.16	.17	.19	.20	.22	.23	.25	.27
5/16	.08	.10	.12	.14	.16	.18	.20	.21	.23	.25	.27	.29	.31	.33
3/8	.09	.12	.14	.16	.19	.21	.23	.26	.28	.30	.33	.35	.38	.40
7/16	.11	.14	.16	.19	.22	.25	.27	.30	.33	.36	.38	.41	.44	.46
1/2	.13	.16	.19	.22	.25	.28	.31	.34	.38	.41	.44	.47	.50	.53
9/16	.14	.18	.21	.25	.28	.32	.35	.39	.42	.46	.49	.53	.56	.60
5/8	.16	.20	.23	.27	.31	.35	.39	.43	.47	.51	.55	.59	.63	.66
11/16	.17	.21	.26	.30	.34	.39	.43	.47	.52	.56	.60	.64	.69	.73
3/4	.19	.23	.28	.33	.38	.42	.47	.52	.56	.61	.66	.70	.75	.80
13/16	.20	.25	.30	.36	.41	.46	.51	.56	.61	.66	.71	.76	.81	.86
7/8	.22	.27	.33	.38	.44	.49	.55	.60	.66	.71	.77	.82	.88	.93
15/16	.23	.29	.35	.41	.47	.53	.59	.64	.70	.76	.82	.88	.94	1.00
1	.25	.31	.38	.44	.50	.56	.63	.69	.75	.81	.88	.94	1.00	1.06
1 1/16	.27	.33	.40	.46	.53	.60	.66	.73	.80	.86	.93	1.00	1.06	1.13
1 1/8	.28	.35	.42	.49	.56	.63	.70	.77	.84	.91	.98	1.05	1.13	1.20
1 3/16	.30	.37	.45	.52	.59	.67	.74	.82	.89	.96	1.04	1.11	1.19	1.26
1 1/4	.31	.39	.47	.55	.63	.70	.78	.86	.94	1.02	1.09	1.17	1.25	1.33
1 5/16	.33	.41	.49	.57	.66	.74	.82	.90	.98	1.07	1.15	1.23	1.31	1.39
1 3/8	.34	.43	.52	.60	.69	.77	.86	.95	1.03	1.12	1.20	1.29	1.38	1.46
1 7/16	.36	.45	.54	.63	.72	.81	.90	.99	1.08	1.17	1.26	1.35	1.44	1.53
1 1/2	.38	.47	.56	.66	.75	.84	.94	1.03	1.13	1.22	1.31	1.41	1.50	1.59
1 9/16	.39	.49	.59	.68	.78	.88	.98	1.07	1.17	1.27	1.37	1.46	1.56	1.66
1 5/8	.41	.51	.61	.71	.81	.91	1.02	1.12	1.22	1.32	1.42	1.52	1.63	1.73
1 11/16	.42	.53	.63	.74	.84	.95	1.05	1.16	1.27	1.37	1.47	1.58	1.69	1.79
1 3/4	.44	.55	.66	.77	.88	.98	1.09	1.20	1.31	1.42	1.53	1.64	1.75	1.86
1 13/16	.45	.57	.68	.79	.91	1.02	1.13	1.25	1.36	1.47	1.59	1.70	1.81	1.93
1 7/8	.47	.59	.70	.82	.94	1.05	1.17	1.29	1.41	1.52	1.64	1.76	1.88	1.99
1 15/16	.48	.61	.73	.85	.97	1.09	1.21	1.33	1.45	1.57	1.70	1.82	1.94	2.06
2	.50	.63	.75	.88	1.00	1.13	1.25	1.38	1.50	1.63	1.75	1.88	2.00	2.13

In calculating the net area add 1/8 inch to diameter of rivet before entering the table.

TABLE XXXIII.

Bridge Pins with Lomas Nuts. American Bridge Company Standards.
All Dimensions in Inches.

Diameter of Pin.	Pin. Screw. Diam.	Pin. Screw. Length.	Pin. Add to Grip.	Standard Dimensions. 6 Threads per Inch.	Nut. Diam. of Rough Hole.	Nut. Short Diam. S	Nut. Long. Diam. L	Nut. Weight in Pounds.	Diameter of Pin.
2	1½	1½	¼		1 5/16	3¼	3¾	2.5	2
2¼	1½	1½	¼		1 5/16	3¼	3¾	2.5	2¼
2½	2	1½	¼		1 13/16	3¾	4 5/16	2.5	2½
2¾	2	1½	¼		1 13/16	3¾	4 5/16	2.5	2¾
3	2½	1½	¼		2 5/16	4½	5 3/16	3.0	3
3¼	2½	1½	¼		2 5/16	4½	5 3/16	3.0	3¼
3½	2½	1½	¼		2 5/16	4½	5 3/16	3.0	3½
3¾	3	1⅞	½		2 13/16	5	5¾	5.5	3¾
4	3	1⅞	½		2 13/16	5	5¾	5.5	4
4¼	3½	1⅞	½		3 5/16	5¾	6⅝	7.0	4¼
4½	3½	1⅞	½		3 5/16	5¾	6⅝	7.0	4½
4¾	3½	1⅞	½		3 5/16	5¾	6⅝	7.0	4¾
5	4	1⅞	½		3 13/16	6½	7½	8.5	5
5¼	4	1⅞	½		3 13/16	6½	7½	8.5	5¼
5½	4½	1⅞	½		4 5/16	7	8⅛	11.0	5½
5¾	4½	1⅞	½		4 5/16	7	8⅛	11.0	5¾
6	4½	1⅞	½		4 5/16	7	8⅛	11.0	6
6¼	5	2⅜	¾		4 13/16	7¾	8 15/16	12.0	6¼
6½	5	2⅜	¾		4 13/16	7¾	8 15/16	12.0	6½
6¾	5½	2⅜	¾		5 5/16	8¼	9½	13.5	6¾
7	5½	2⅜	¾		5 5/16	8¼	9½	13.5	7
7½	5½	2⅜	¾		5 5/16	8¼	9½	13.5	7½
7¾	6	2⅜	¾		5 13/16	9	10⅜	17.0	7¾
8	6	2⅜	¾		5 13/16	9	10⅜	17.0	8

Note.—To obtain grip "G" add 1/16" for each bar, together with amount given in table.

placed in the hole. Bolts (b) and (c) are placed in the hole which is then filled with Portland cement, sulphur or lead. Where anchor bolts are to take direct stress they should be made long enough so that the weight of the masonry will more than equal the strength of the bolt. A usual specification for anchorage is that the weight of the masonry shall be one and one-half times the anchoring stress.

TABLE XXXIV.

MAXIMUM ALLOWABLE BENDING MOMENTS IN PINS FOR VARIOUS FIBER STRESSES.

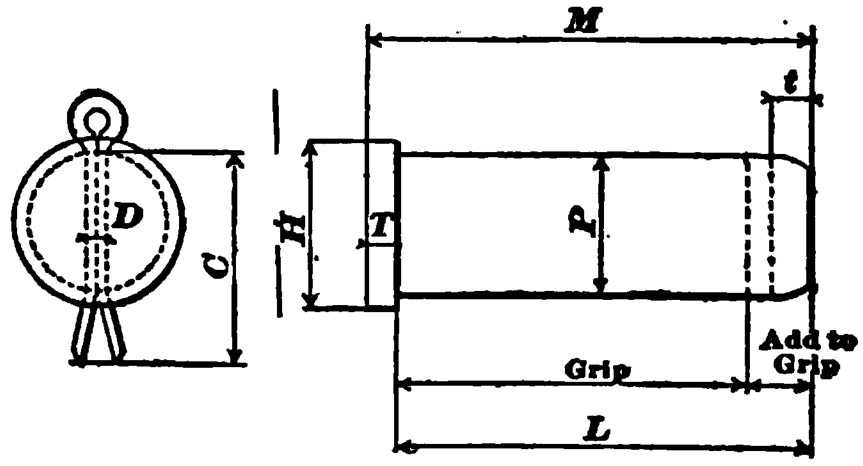
Pin.		Moments in Inch Pounds for Fiber Stresses per Square Inch of				
Diam. Ins.	Area.	15,000	18,000	20,000	22,000	25,000
1	0.785	1,470	1,770	1,960	2,160	2,450
1¼	1.227	2,880	3,450	3,830	4,220	4,790
1½	1.767	4,970	5,960	6,630	7,290	8,280
1¾	2.405	7,890	9,470	10,500	11,570	13,200
2	3.142	11,800	14,100	15,700	17,280	19,600
2¼	3.976	16,800	20,100	22,400	24,600	28,000
2½	4.909	23,000	27,600	30,700	33,700	38,400
2¾	5.940	30,600	36,800	40,800	44,900	51,000
3	7.069	39,800	47,700	53,000	58,300	66,300
3¼	8.296	50,600	60,700	67,400	74,100	84,300
3½	9.621	63,100	75,800	84,200	92,600	105,200
3¾	11.045	77,700	93,200	103,500	113,900	129,400
4	12.566	94,200	113,100	125,700	138,200	157,100
4¼	14.186	113,000	135,700	150,700	165,800	188,400
4½	15.904	134,200	161,000	178,900	196,800	223,700
4¾	17.721	157,800	189,400	210,400	231,500	263,000
5	19.635	184,100	220,900	245,400	270,000	306,800
5¼	21.648	213,100	255,700	284,100	312,500	355,200
5½	23.758	245,000	294,000	326,700	359,300	408,300
5¾	25.967	280,000	335,900	373,300	410,600	466,600
6	28.274	318,100	381,700	424,100	466,500	530,200
6¼	30.680	359,500	431,400	479,400	527,300	599,200
6½	33.183	404,400	485,300	539,200	593,100	674,000
6¾	35.785	452,900	543,500	603,900	664,200	754,800
7	38.485	505,100	606,100	673,500	740,800	841,900
7¼	41.282	561,200	673,400	748,200	823,000	935,300
7½	44.179	621,300	745,500	828,400	911,200	1,035,400
7¾	47.173	685,500	822,600	914,000	1,005,300	1,142,500
8	50.265	754,000	904,800	1,005,300	1,105,800	1,256,600
8¼	53.456	826,900	992,300	1,102,500	1,212,800	1,378,200
8½	56.745	904,400	1,085,200	1,205,800	1,326,400	1,507,300
8¾	60.132	986,500	1,183,800	1,315,400	1,446,900	1,644,200

PINS.—The American Bridge Company's standard bridge pins with Lomas nuts are given in Table XXXIII. Square nuts are sometimes used. The figured grip for Lomas nuts is increased as shown to make sure that the pin has a full bearing. Where square nuts are used a washer should be provided at one end and the grip should be

TABLE XXXV.

AMERICAN BRIDGE COMPANY STANDARD COTTER PINS.

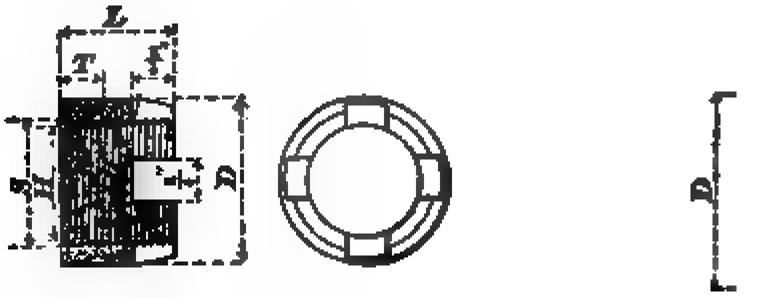
All dimensions in inches.



Diam. of Pin. P	Pin. Diameter of Pin-hole.	Pin. Taper at End. t	Head. Diameter H	Head. Thickness T	Cotter. Length. C	Cotter. Diameter. D	Add to Grip. For Length Over All. M	Add to Grip. For Length Under Head. L	Diam. of Pin. P
1	$1\frac{1}{32}$	$\frac{5}{16}\times\frac{1}{16}$	$1\frac{1}{4}$	$\frac{1}{4}$	$1\frac{3}{4}$	$\frac{1}{4}$	$\frac{7}{8}$	$\frac{5}{8}$	1
$1\frac{1}{4}$	$1\frac{9}{32}$	$\frac{5}{16}\times\frac{1}{16}$	$1\frac{1}{2}$	$\frac{1}{4}$	2	$\frac{1}{4}$	$\frac{7}{8}$	$\frac{5}{8}$	$1\frac{1}{4}$
$1\frac{1}{2}$	$1\frac{17}{32}$	$\frac{7}{16}\times\frac{3}{32}$	$1\frac{3}{4}$	$\frac{1}{4}$	$2\frac{1}{2}$	$\frac{5}{16}$	$1\frac{1}{8}$	$\frac{7}{8}$	$1\frac{1}{2}$
$1\frac{3}{4}$	$1\frac{25}{32}$	$\frac{7}{16}\times\frac{3}{32}$	2	$\frac{1}{4}$	$2\frac{3}{4}$	$\frac{5}{16}$	$1\frac{1}{8}$	$\frac{7}{8}$	$1\frac{3}{4}$
2	$2\frac{1}{32}$	$\frac{1}{2}\times\frac{1}{8}$	$2\frac{3}{8}$	$\frac{3}{8}$	3	$\frac{3}{8}$	$1\frac{3}{8}$	1	2
$2\frac{1}{4}$	$2\frac{9}{32}$	$\frac{1}{2}\times\frac{1}{8}$	$2\frac{5}{8}$	$\frac{3}{8}$	$3\frac{1}{4}$	$\frac{3}{8}$	$1\frac{3}{8}$	1	$2\frac{1}{4}$
$2\frac{1}{2}$	$2\frac{17}{32}$	$\frac{5}{8}\times\frac{5}{32}$	$2\frac{7}{8}$	$\frac{3}{8}$	$3\frac{3}{4}$	$\frac{7}{16}$	$1\frac{1}{2}$	$1\frac{1}{8}$	$2\frac{1}{2}$
$2\frac{3}{4}$	$2\frac{25}{32}$	$\frac{5}{8}\times\frac{5}{32}$	$3\frac{1}{8}$	$\frac{3}{8}$	4	$\frac{7}{16}$	$1\frac{1}{2}$	$1\frac{1}{8}$	$2\frac{3}{4}$
3	$3\frac{1}{32}$	$\frac{3}{4}\times\frac{3}{16}$	$3\frac{1}{2}$	$\frac{1}{2}$	5	$\frac{1}{2}$	$1\frac{7}{8}$	$1\frac{3}{8}$	3
$3\frac{1}{4}$	$3\frac{9}{32}$	$\frac{3}{4}\times\frac{3}{16}$	$3\frac{3}{4}$	$\frac{1}{2}$	5	$\frac{1}{2}$	$1\frac{7}{8}$	$1\frac{3}{8}$	$3\frac{1}{4}$
$3\frac{1}{2}$	$3\frac{17}{32}$	$\frac{7}{8}\times\frac{7}{32}$	4	$\frac{1}{2}$	6	$\frac{5}{8}$	$2\frac{1}{8}$	$1\frac{5}{8}$	$3\frac{1}{2}$
$3\frac{3}{4}$	$3\frac{25}{32}$	$\frac{7}{8}\times\frac{7}{32}$	$4\frac{1}{4}$	$\frac{1}{2}$	6	$\frac{5}{8}$	$2\frac{1}{8}$	$1\frac{5}{8}$	$3\frac{3}{4}$

Note.—Use pins with lomas nuts, in preference to cotter pins, whenever possible.

increased accordingly. In calculating the grip it is usual to assume that bars may be $\frac{1}{16}$ inch thicker than the figured thickness, that riveted members may be $\frac{1}{4}$ inch wider or narrower than the figured dimensions. Members should be packed on the pin so that the bending moments will be as small as possible. Steel pilot nuts and points for protecting the threads of the pin are shown in Fig. 183*a* and Fig. 183*b*, respect-



(*a*) PILOT NUT. (*b*) PILOT POINT.

FIG. 183. AMERICAN BRIDGE COMPANY PILOT NUTS AND POINTS.

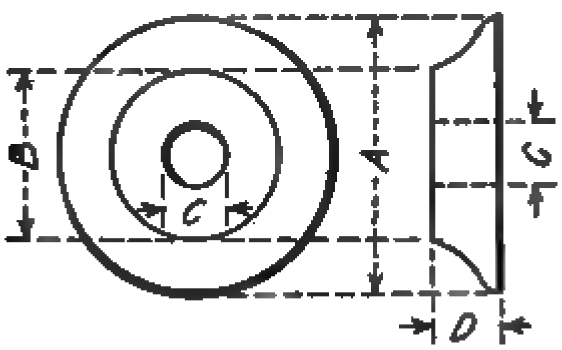
ively. The allowable bending moments on pins for different fiber stresses are given in Table XXXIV. The method of calculating the stresses in pins is described in detail in Chapter VIII.

Lateral Pins.—The American Bridge Company's standard cotter pins are given in Table XXXV. These pins are used only for laterals and other similar members.

CAST WASHERS.—The weight of cast O G washers are given in Table XXXVI.

TABLE XXXVI.

STANDARD CAST, O G WASHERS.



Diameter of Bolt = d

$A = 4d + \frac{1}{4}''$, $C = d + \frac{1}{8}''$
$B = 2d + \frac{1}{2}''$, $D = d$. } *For sizes not given below.*

Diameter of Bolt *d*	*A*	*B*	*C*	*D*	Weight in Pounds.
½	2⅝	1¼	9/16	⅝	½
⅝	3	1⅞	11/16	¾	¾
¾	3¼	2⅛	13/16	⅞	1¼
⅞	3¾	2½	15/16	⅞	1½
1	4	2¾	1 1/16	1⅛	2⅝
1⅛	4¾	2¾	1 3/16	1⅛	3
1¼	6	3	1 5/16	1¼	5¾
1½	6¼	3¼	1⅝	1½	6
1¾	7¼	3¾	1⅞	1¾	9⅝
2	8¼	4¼	2⅛	2	17¾

CHAPTER XIV.

THE DETAILS OF HIGHWAY BRIDGE MEMBERS.

FLOORBEAMS.—Floorbeams for highway bridges are made of rolled **I** beams or riveted plate girders. Joists or stringers are made either of rolled **I** beams or of timber, wooden beams being now used only for unimportant country bridges. Stringers on electric railway bridges are sometimes made of plate girders. Floorbeams may be riveted to the posts or hung from the lower chord pins. The floorbeams of deck bridges are sometimes carried directly on the top chords. Several different types of floorbeam connection are shown in Fig. 184. Floorbeams may be riveted above the lower chord or may be suspended below the lower chord. The first method gives the most rigid structure, while the second method is the cheapest type of construction. The rod hanger in (a) was formerly much used for all classes of highway bridges, but is now seldom used except for light country bridges. Plate hanger (b) is used for the floorbeam at the foot of the hip vertical, while (c) is used for the floorbeams at intermediate posts. Unless great care is used in the design and erection, suspended floorbeams are liable to lack rigidity. The riveted connections in (d) and (f) are cheap and effective. The connection in (e) is eccentric but is quite effective. Connection (h) is very satisfactory. Connection (g) is a special connection. Details of floorbeams riveted above the lower chords are shown in Figs. 148, 150, 159, 161 and 169; while details of floorbeams riveted below the chords are shown in Figs. 151, 154, 157, 158, 163, 166 and 285. Details of riveted plate girder floorbeams are given in Fig. 185. The details of a lattice floorbeam are given in Fig. 186.

Details of sidewalk brackets are shown in Figs. 185, 186, 193 and 194. This detail should be designed with care.

Joists may be carried directly on the tops of the floorbeams or on shelf angles riveted to the web plates of the floorbeams. For timber floor plank, joists should not be spaced at greater distances than one foot

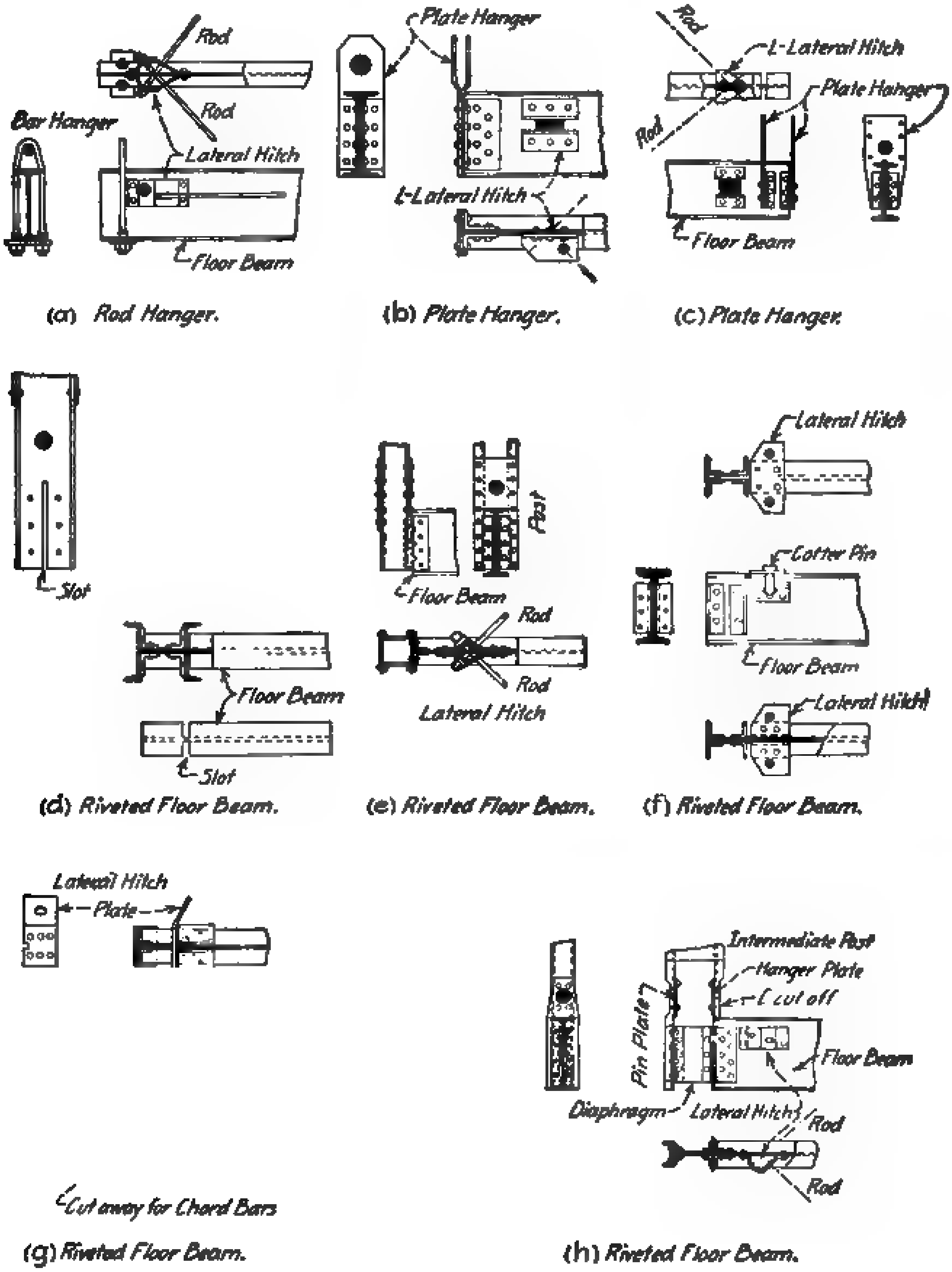


FIG. 184. FLOORBEAM CONNECTIONS FOR HIGHWAY BRIDGES.

for each inch in thickness of the floor plank, with a maximum spacing of three feet. Joists for other coverings are spaced as required.

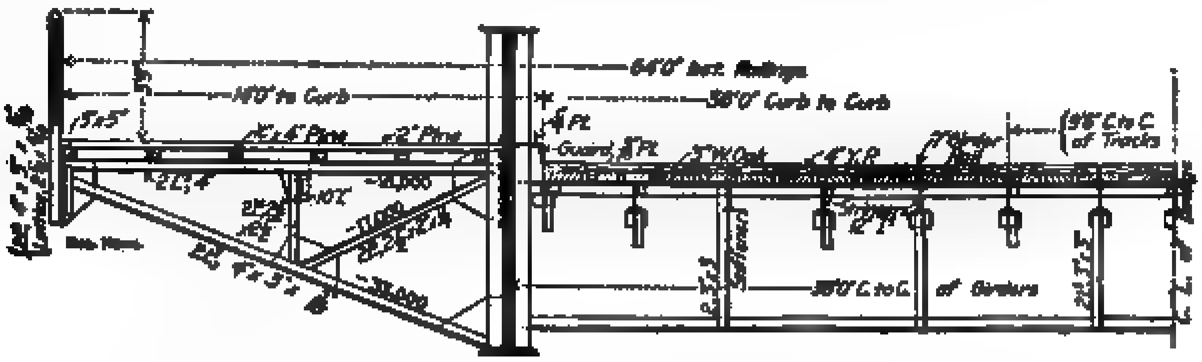


FIG. 185. PLATE GIRDER FLOORBEAM AND SIDEWALK BRACKET.

HIGHWAY BRIDGE FLOORS.—Highway bridge floors are made of timber as in Figs. 141, 145, 158, 162 and 285; reinforced concrete as in Figs. 143, 154, 188, 191 and 192; buckle plate floor as in Figs. 169 and 186; corrugated steel floor, Z-bar floor, or angle and plate floor filled with concrete as in Fig. 190; Buckeye steel floor as in (a) Fig. 192; or Multiplex steel floor as in (b) Fig. 192. For specifications for floors, see Chapter II and Appendix I.

FIG. 186. LATTICE FLOORBEAM AND SOLID BUCKLE PLATE FLOOR.

Plank Floors.—The wearing surface should be of white oak or similar timber laid transversely of the bridge. Where two layers of plank are used the lower layer is commonly laid diagonally. Planks should be laid from $\frac{1}{4}$ to $\frac{1}{2}$ inch apart so that water will not be retained but will run through and give the planks an opportunity to dry out. Where more than one layer of planks is used the moisture is retained and decay is quite rapid unless precautions are taken to protect the

timber. A liberal application of coal tar to the surfaces that are not exposed will prolong the life of the floor materially. If possible, timber treated by a preservative process should be used for floors composed of more than one layer of plank. Each plank should be solidly spiked to the joists, using 40*d* spikes for planks 2½ inches thick and under, and 60*d* spikes for planks from 2¾ to 4 inches thick. Where steel joists are used, spiking strips about 3″ × 8″ are bolted to the tops of all joists, or spiking strips 4″ × 6″ are bolted to the sides of three lines of joists under each plank length. When the latter method is used the floor planks are fastened to the intermediate joists by bending spikes, driven through the floor plank, around the upper flanges of the joists. Two channels with a 3″ × 6″ spiking piece are sometimes used for the center line of joists. Channels are commonly used for the outside joists. For specifications for plank floors, see Chapter II and Appendix I.

Reinforced Concrete Floors.—Reinforced concrete floors are designed as in Chapter XVII. The lower edge is reinforced with expanded

FIG. 187. DETAILS OF A RIVETED FLOORBEAM.

metal as in Figs. 143 and 191; with rods as in Fig. 154, or with wire netting or other form of reinforcement. The wearing surface of the roadway of reinforced concrete floors is commonly made of asphalt as in Fig. 191, of paving brick as in Fig. 172, of wooden blocks or other form of pavement. Recently the wearing surface has been made of roughened concrete, but its use is still in the experimental stage.

(Roughened concrete wearing surfaces on bridges in Denver, Colo., are giving excellent satisfaction.) The wearing surfaces of footwalks are made of concrete with a cement finish as for sidewalks. Care

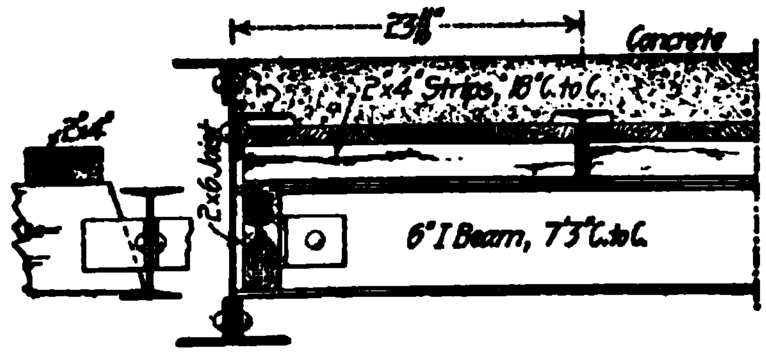


FIG. 188. CONCRETE SIDEWALK FLOOR.

should be used to provide an expansion joint at one end (at the expansion end) of the bridge. This may be accomplished by means of a bent plate filled with asphalt placed between the end of the floor and the abutment, or as in Fig. 191*b*.

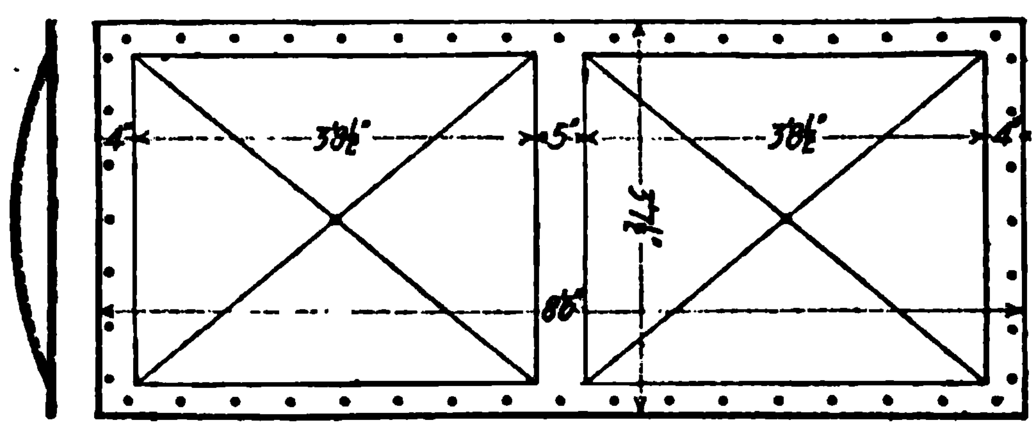


FIG. 189. BUCKLE PLATES.

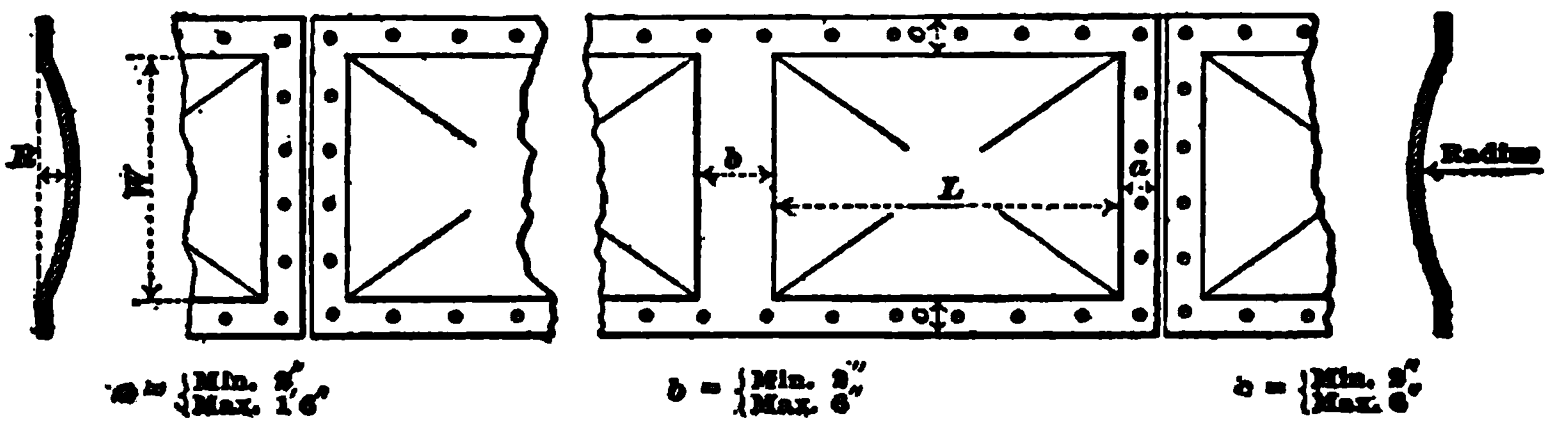
Buckle Plates.—Buckle plates are made by "dishing" flat plates as in Table XXXVII. The width of the buckle *W* or length *L*, varies from 2′ 6″ to 5′ 6″. The buckles may be turned with the greater dimension in either direction of the plate. Several buckles may be put

FIG. 190. CORRUGATED STEEL FLOORS.

in one plate, all of which must be the same size and symmetrically placed. Buckle plates are made $\frac{1}{4}$″, $\frac{5}{16}$″, $\frac{3}{8}$″ and $\frac{7}{16}$″ in thickness. The common standard sizes are given in Table XXXVII.

TABLE XXXVII.

AMERICAN BRIDGE COMPANY'S STANDARD BUCKLE PLATES.



No. of Plate.	Size of Buckle in Feet and Inches.		Rise in Inches.	Rad. of Buckle in Feet and Inches.		Maximum No. of Buckles.	No. of Plate.	Size of Buckle in Feet and Inches.		Rise in Inches.	Rad. of Buckle in Feet and Inches.		Maximum No. of Buckles.
	Length. L	Width. W	R	Length. L	Width. W			Length. L	Width. W	R	Length. L	Width. W	
1	3-11	4-6	3½	6-8	8-9	7	20	2-9	2-6	2½	4-7	3-10	10
2	4-6	3-11	3½	8-9	6-8	6	21	2-6	2-6	2½	3-10	3-10	10
3	3-11	3-6	3	7-9	6-3	7	22	3-5	3-6	3	5-11	6-3	8
4	3-6	3-11	3	6-3	7-9	8	23	3-6	3-5	3	6-3	5-11	8
5	3-9	3-9	3	7-1	7-1	8	24	3-6	3-9	3	6-3	7-1	8
6	3-1	3-9	3	4-10	7-1	9	25	3-9	3-6	3	7-1	6-3	8
7	3-9	3-1	3	7-1	4-10	8	26	3-1	3-2	3	4-10	5-1	9
8	3-8	3-8	2	10-2	10-2	8	27	3-2	3-1	3	5-1	4-10	9
9	2-8	3-8	2	5-5	10-2	10	28	3-1	3-0	3	4-10	4-7	9
10	3-8	2-8	2	10-2	5-5	8	29	3-0	3-1	3	4-7	4-10	9
11	2-2	3-8	2	3-7	10-2	10	30	2-0	2-6	2½	2-6	3-10	10
12	3-8	2-2	2	10-2	3-7	8	31	2-6	2-0	2½	3-10	2-6	15
13	3-0	3-0	2	6-10	6-10	9	32	3-6	5-6	3½	5-4	13-1	5
14	2-9	2-9	3	3-10	3-10	10	33	5-6	3-6	3½	13-1	5-4	1
19	2-6	2-9	2½	3-10	4-7	10	34	4-0	4-0	3	8-1	8-1	7

Plates are made ¼″, $\frac{5}{16}$″, ⅜″ or $\frac{7}{16}$″ thick.
Buckles of different sizes should not be used in the same plate.
Rivets generally ⅝″ or ¾″ diameter.

Buckle plates should be firmly bolted or riveted around the edges with a maximum spacing of 6 inches, and should be supported transversely between the buckles. The process of buckling distorts the plate and an extra width should be ordered, and the plate trimmed after the process is complete.

Strength of Buckle Plates.—The safe load for a buckle plate with buckles placed up, is approximately given by the formula

$$W = 4f \cdot R \cdot t$$

where W = total safe uniform load;

f = safe unit stress in pounds per square inch;

R = depth of buckle in inches;

t = thickness of plate in inches.

Where buckle plates are riveted and the buckles placed down, the safe load is from 3 to 4 times that given above.

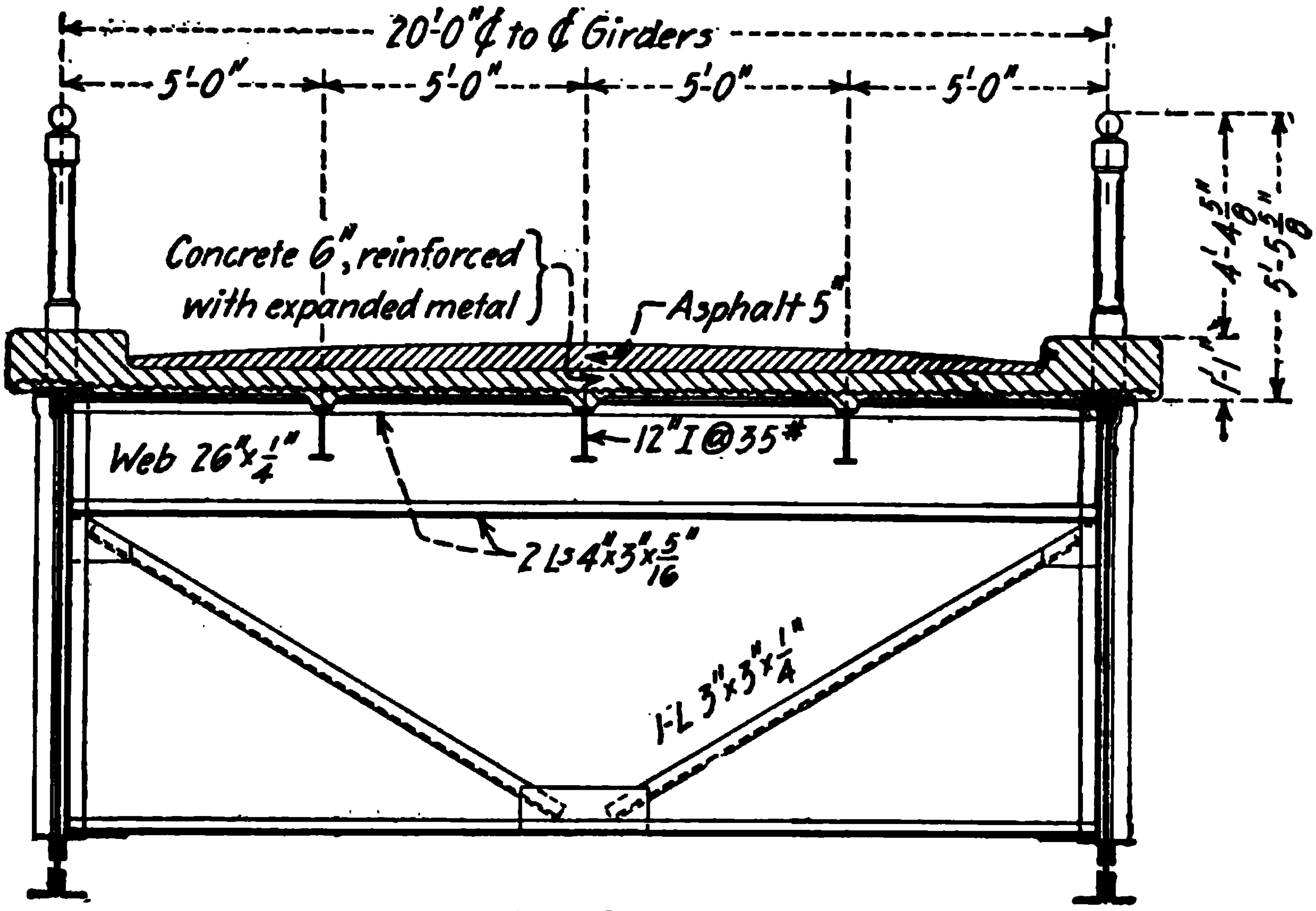


FIG. 191*a*. FLOOR OF HARRISON STREET PLATE GIRDER BRIDGE, PRINCETON, N. J. (AMERICAN BRIDGE CO.)

Buckle plates are usually fastened to the tops of the stringers as in Fig. 186, but may be fastened to the bottom flanges as in Fig. 194. The space above the buckle plates may be filled with asphalt as in Fig. 186, or with concrete with a finished pavement as in Fig. 194.

Corrugated Steel Floors.—The steel floors shown in Fig. 190 are used on heavy city and on railroad bridges. The space above the plates is filled with either asphalt or concrete, and a covering is used as for concrete floors. For electric and steam railways the ties are sometimes

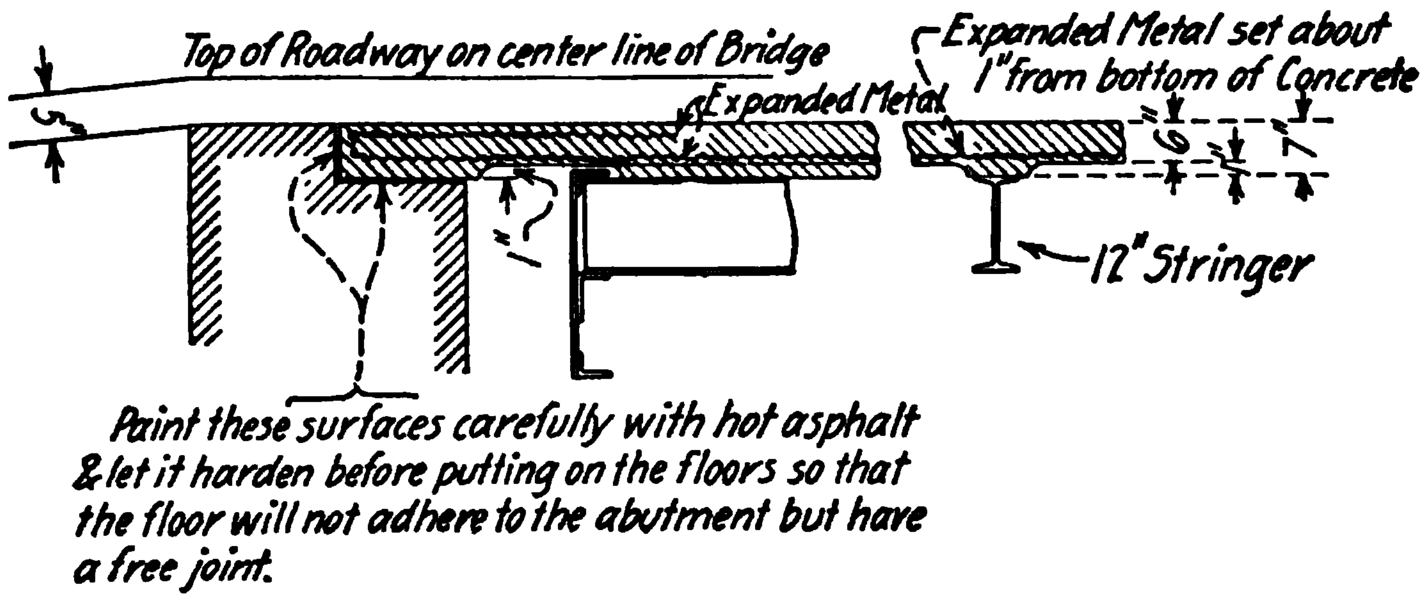


FIG. 191*b*. DETAIL OF FLOOR OF HARRISON STREET PLATE GIRDER BRIDGE, PRINCETON, N. J. (AMERICAN BRIDGE CO.)

laid on ballast in the troughs. The details, weights and safe loads for corrugated plates are given in Pencoyd Iron Works' handbook, in Carnegie Steel Company's handbook, and in Trautwine's Pocket-book. Details of corrugated plates are also given in the American Bridge Company's "Standards for Structural Details."

Corrugated flooring or trough plates are usually very hard to get and the Z-bar and plate floor shown in (c) Fig. 190, and the angle and plate floor shown in (d) are substituted. The details, weights and safe loads for Z-bar and plate flooring are given in the handbooks above named. Angle and plate flooring is made of equal-legged angles and plates, and the safe loads are not given in the handbooks but must be calculated. The moment of inertia, I, of a section of flooring containing two angles and two plates is given by the formula

$$I = 2I' + 2Ad^2 + 2I''$$

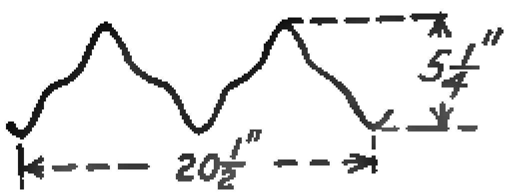
where I' = moment of inertia of one angle about an axis through the center of gravity of the angle parallel to the neutral axis of the flooring; A = area of one angle; d = distance from center of gravity of the angle to the neutral axis of the flooring; I'' = moment of inertia of the plate about the neutral axis.

The properties of the angles required in the calculations may be obtained from the handbooks, and I'' is equal to one-half the sum of the moments of inertia of the plate about its long and its short diam-

eter—since the sum of the moments of inertia about any pair of rectangular axes is a constant.

"Buckeye" Steel Flooring.—The corrugated steel floor manufactured by the Youngstown Iron and Steel Roofing Co., Youngstown, Ohio, is shown in (a) Fig. 192. The space above the troughs is filled with asphalt or with concrete as in other forms of steel flooring. The weights and safe loads are given in the manufacturer's catalog.

"Multiplex" Steel Flooring.—The corrugated steel floor manufactured by the Berger Mfg. Co., Canton, Ohio, is shown in (b) Fig. 192. The space above the troughs is filled with asphalt or concrete. The weights and safe loads are given in the manufacturer's catalog.



"BUCKEYE" FLOORING ON BRIDGES (a)

MULTIPLEX STEEL PLATE FLOOR (b)

Fig. 192.

PORTALS.—Portals for bridges should be designed so as to be simple in makeup and with statically determinate stresses. The most common types of portals are shown in Fig. 99 and Fig. 101 in Chapter VII. The portals shown in (a) and (b) Fig. 99, and in Fig. 101, are the types most commonly used. Details of portals are shown in Figs. 163, 166, 167, 169 and 285, while views of portals are shown in Figs. 10, 16 and 165. The American Bridge Company's standard portal for highway bridges is shown in Fig. 16, although this company probably builds more bridges with portals like (a) Fig. 99. The portal bracing

should either be placed in the plane of the center line of the end-posts, as in Figs. 16 and 163, or the portal should be box-laced with the same

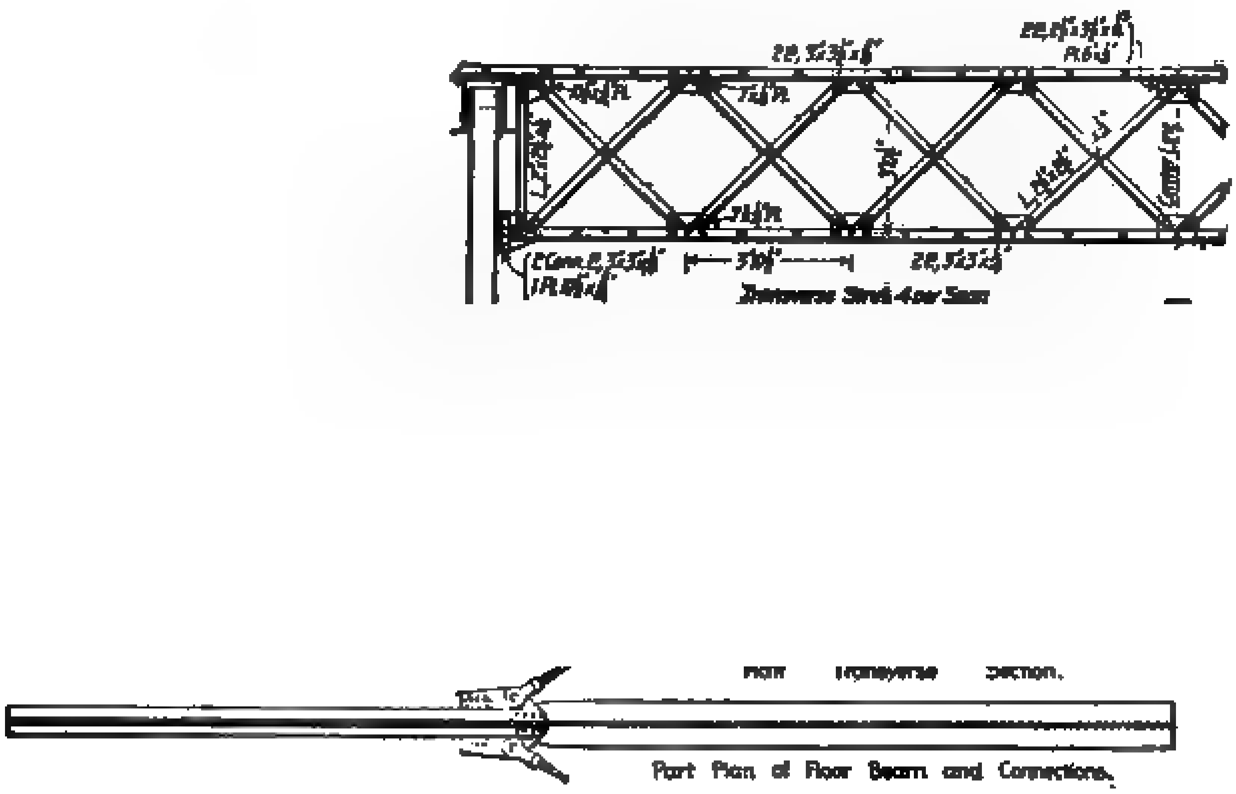


Fig. 193. Cross-section of Highway Bridge.

Fig. 194. Details of Concrete Floor.

depth as the posts at right angles to the plane of the end-posts. The top portal strut should be fastened as in Figs. 16, 163, or 285, and in no case should it be fastened directly to the chord pin U_1.

Portal bracing should always be made of angles or other sections with riveted connections. Where cover plates are not used, the end-posts should be strengthened with diaphragm plates at the foot of the portal knee braces.

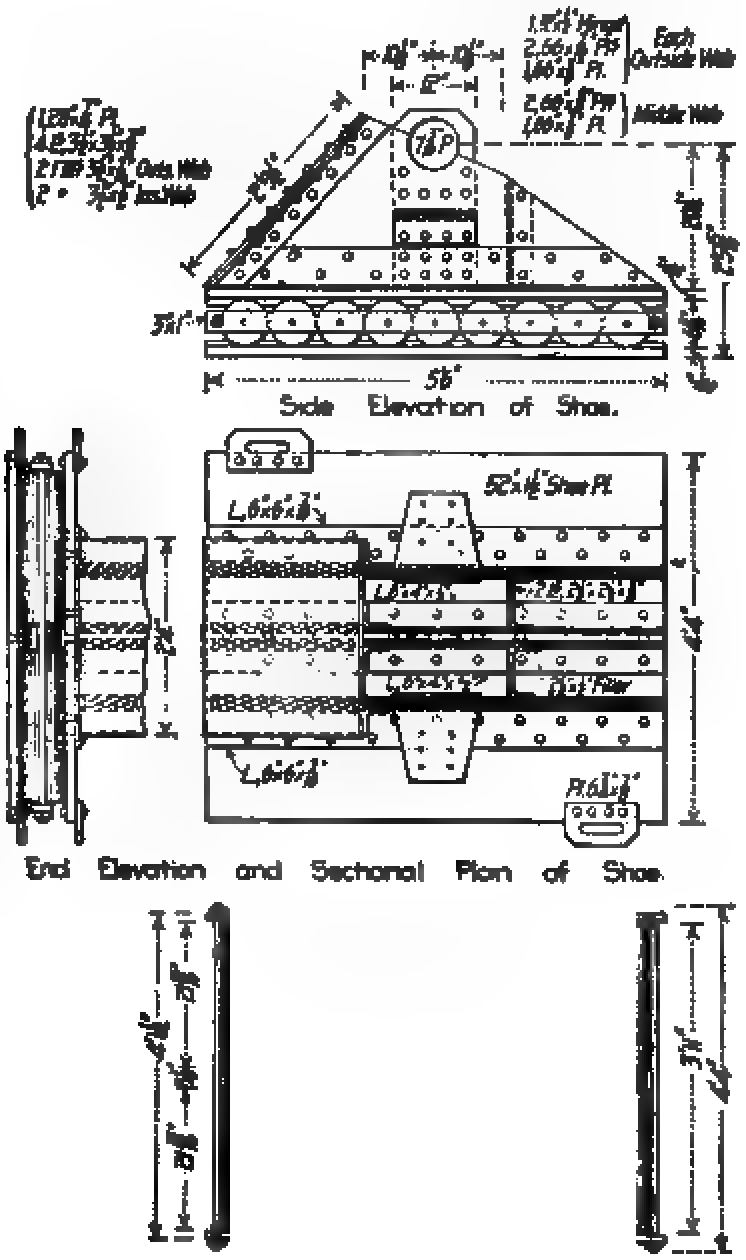


FIG. 195. DETAILS OF SHOE AND ROLLER NEST FOR THE HIGHWAY BRIDGE IN FIG. 169.

The members in a bridge portal are subject to reversals of stress, and this should be considered in the design. The best section for portals of highway bridges is two unequal-legged angles, placed back to back with the long legs placed outward.

Sway Bracing.—Sway bracing is similar to portal bracing and should be designed in the same manner. Where the head room is lim-

ited knee-braced lateral struts are sometimes substituted for sway bracing, as in (f) Fig. 99. The analysis of this problem is given in Chapter VII.

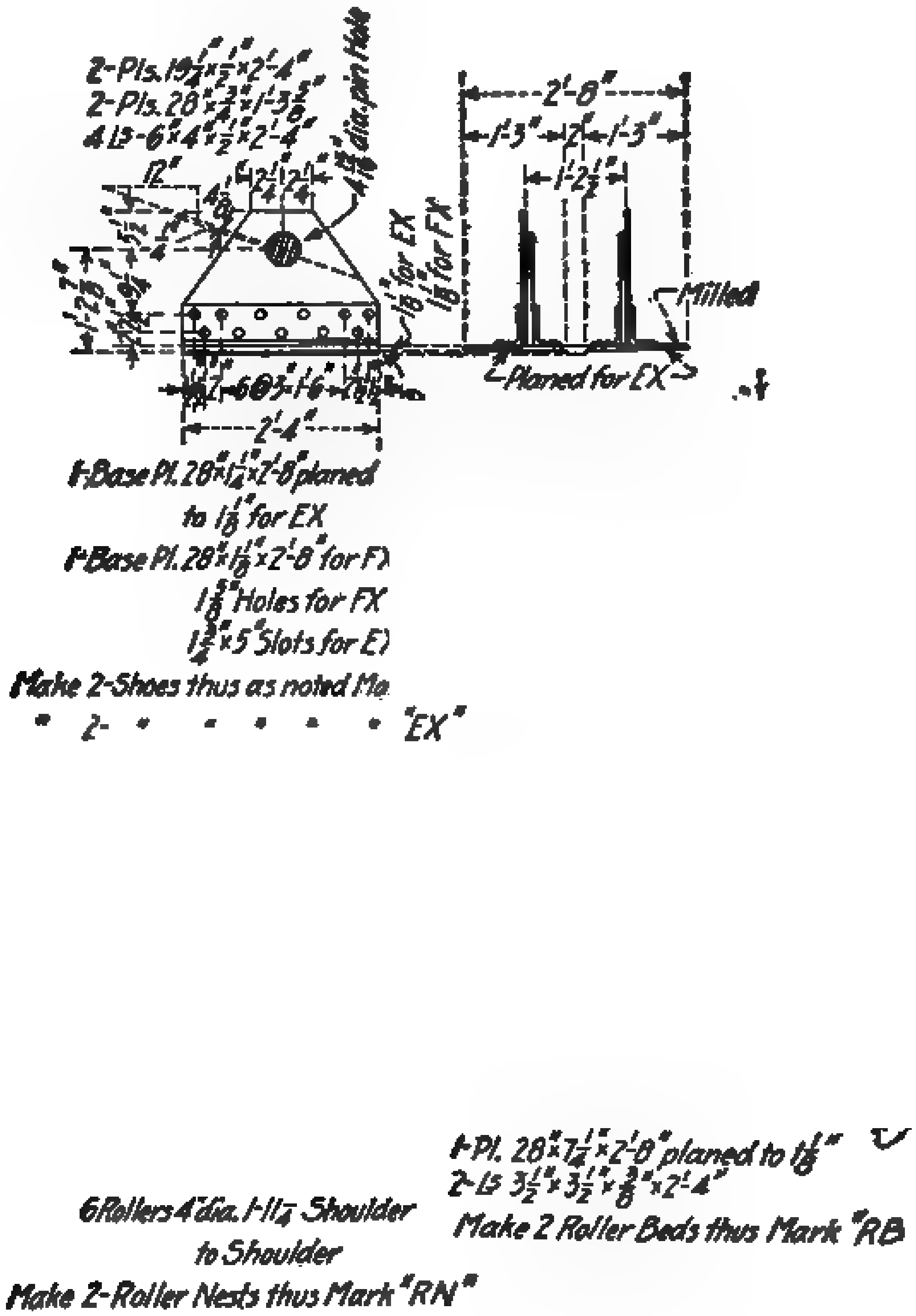


FIG. 196. DETAILS OF SHOES AND ROLLER NEST FOR A HIGHWAY BRIDGE.

SHOES AND PEDESTALS.—The bridge rests on shoes or pedestals, the loads being transferred to the shoes in pin-connected bridges by means of pins, and through the riveted joints in riveted bridges.

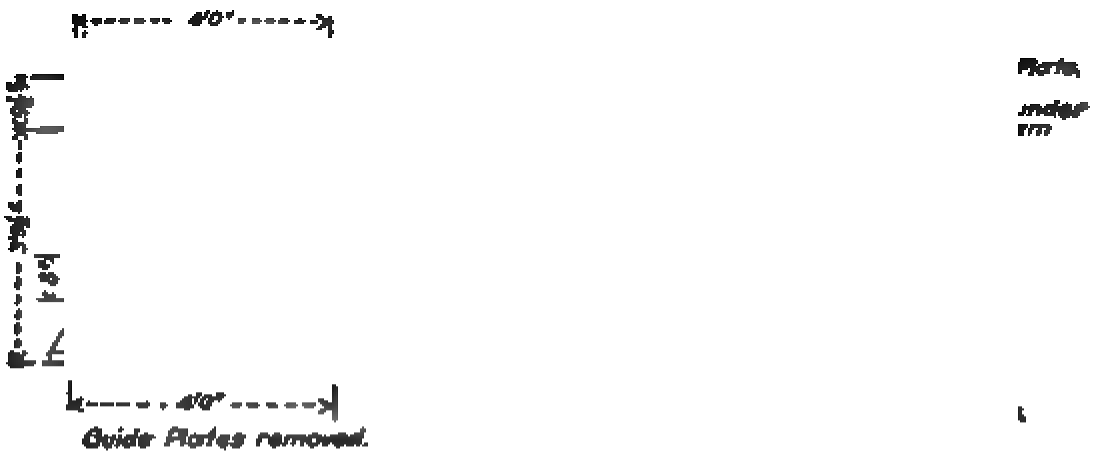
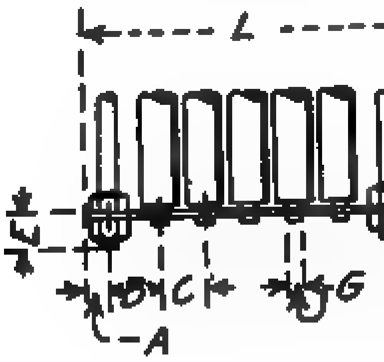


FIG. 197. SEGMENTAL ROLLERS.

TABLE XXXVIII.

ROLLER NESTS, YOUNGSTOWN BRIDGE COMPANY STANDARDS.



Number.	Diameter. Ins.	Number of Rollers.	Bar. Ins.	A Ins.	B Ins.	C Ins.	Total Length L. Ins.	Diameter of Rod. Ins.	2 E Ins.	G Ins.
5	2½	5	2 x 5/16	1¼	2½	2¾	18½	⅞	2¾	1¼
6	2½	6	2 x 5/16	1¼	2½	2¾	21¼	⅞	2¾	1¼
7	3	5	2½ x 5/16	1¼	2¾	3¼	21	1	3	1½
8	3	6	2½ x 5/16	1¼	2¾	3¼	24¼	1	3	1½
9	3½	6	3 x ⅜	1¼	3	3¾	27¼	1	3¼	1¾
10	3½	7	3 x ⅜	1¼	3	3¾	31	1	3¼	1¾

The shoes at the expansion ends of the bridge are placed on smooth, sliding plates for bridges of less than, say, 70 feet span, and on nests of rollers for spans of greater length. The action of the rollers under the expansion ends of riveted bridges will be much more satisfactory if the shoes are pin-connected to the truss the same as for pin-connected trusses. Rollers should be made with as large diameters as practicable in order to reduce the pressure on the base plate and also to reduce the resistance to movement. Experience shows that even for light

bridges rollers smaller than 3" diameter are practically worthless. To economize space, segmental rollers, as shown in Fig. 197, are often used for heavy spans.

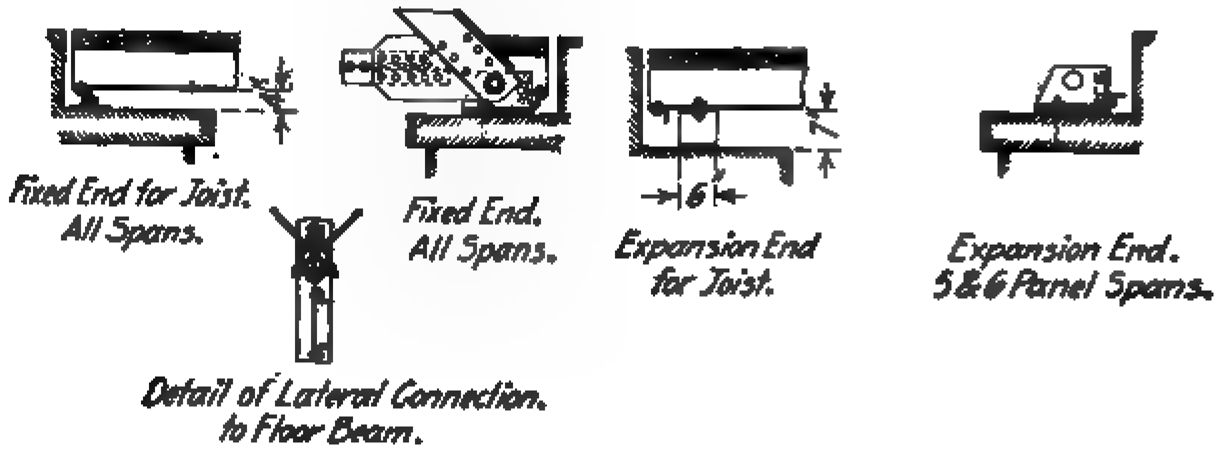


FIG. 198. SHOES FOR RIVETED HIGHWAY BRIDGE. (AMERICAN BRIDGE CO.)

It is usual to specify that a movement produced by a variation of 150 degrees Fahr. be provided for. The coefficient of expansion of steel is approximately 0.0000067 per degree Fahr., which makes it necessary to provide for approximately one inch of movement for each 80 feet of bridge span.

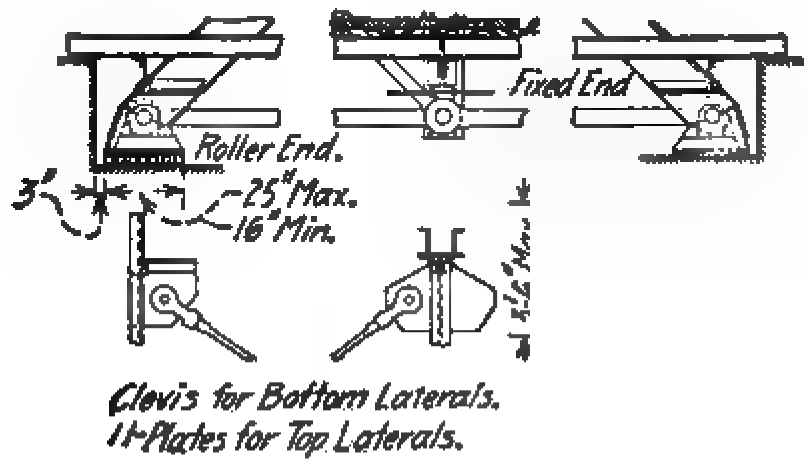


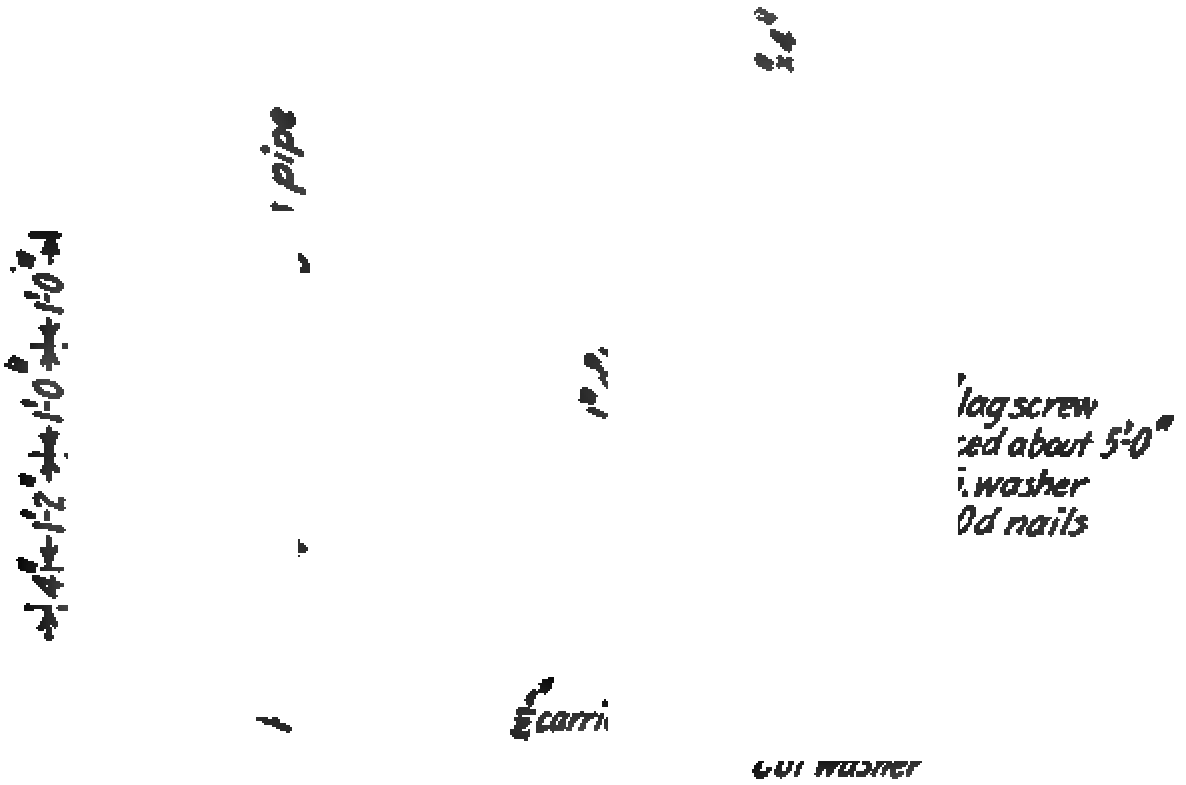
FIG. 199. SHOES AND FLOOR DETAILS FOR PIN-CONNECTED HIGHWAY BRIDGES. (AMERICAN BRIDGE CO.)

Details of shoes and rollers for highway spans are shown in Figs. 195 and 196. Other details are shown in Figs. 163 and 285. Where both bridge seats are of the same height, the fixed end is carried on cast iron pedestal blocks. The blocks are usually made with recesses (honey-combed) to reduce the weight. The shoes and other details of the American Bridge Company's standard highway bridges are shown

in Fig. 198 and Fig. 199. For the calculation of stresses in rollers, see Chapter VIII.

The Youngstown Bridge Company's standard roller nests for highway bridge spans are given in Table XXXVIII.

FENCE AND HUB GUARDS.—The simplest form of fence is that shown in (b) Fig. 200. The posts are made of 4″ × 4″ pieces and



(a) (b)

Fig. 200.

are spaced about 8 feet apart. The top railing is made of two pieces 2″ × 4″, while the side piece is a 2″ × 8″. Similar details are used for steel joists. The 4″ × 6″ felloe guard should be firmly bolted to the floor. Blocks of wood 1 or 2 inches thick, called "shims," are sometimes placed between the felloe guard and the floor. Shims are of questionable utility, and should not be used. A gas pipe fence with angle rail is shown in (a) Fig. 200.

A gas pipe railing with angle posts is shown in Fig. 141; while a gas pipe railing with gas pipe posts is shown in Fig. 201. The posts should be spaced not more than 8 feet apart. The rail in Fig. 201 was used in the Pennsylvania Ave. Subway, Philadelphia, and was furnished at $0.95 per lineal foot. An ornamental fence with pipe top

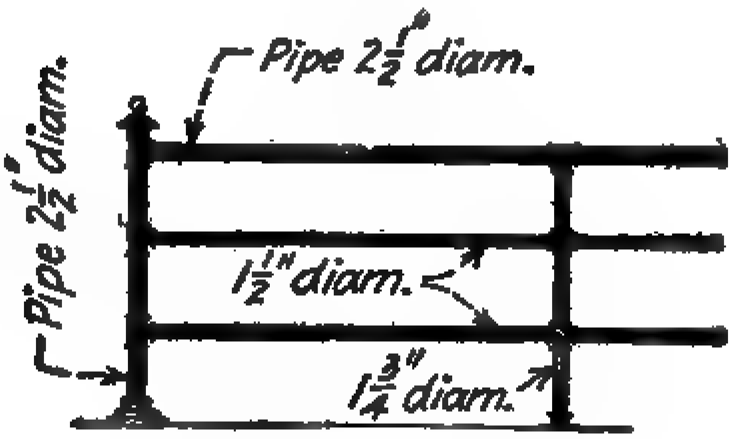


FIG. 201.

FIG. 202.

FIG. 203.

rail and cast iron newel posts is shown in Fig. 202. This rail was used on the same contract as that shown in Fig. 201, and was furnished at \$2.00 per lineal foot. The fence shown in Fig. 203 and the fence

shown in Fig. 204 are good examples of elaborate and expensive fences. The fence shown in Fig. 205 was used on the bridge shown in Fig. 169.

FIG. 204.

FIG. 205.

CRESTINGS.—Crestings are sometimes used on the tops of portals of through bridges. The crestings shown in Fig. 206 will give an idea of this detail. Crestings are also made of cast iron.

FIG. 206. CRESTING.

LATERAL CONNECTIONS.—The connections of riveted laterals are made by riveting to lateral plates in the lower lateral system

and by riveting directly to the tops of the chords in the upper lateral system. Where rods are used for laterals special connections are required.

Upper Lateral Connections.—The King Bridge Company's standard upper lateral connections are given in Figs. 207, 208 and 209. These connections are riveted to the top chords of the bridge. This

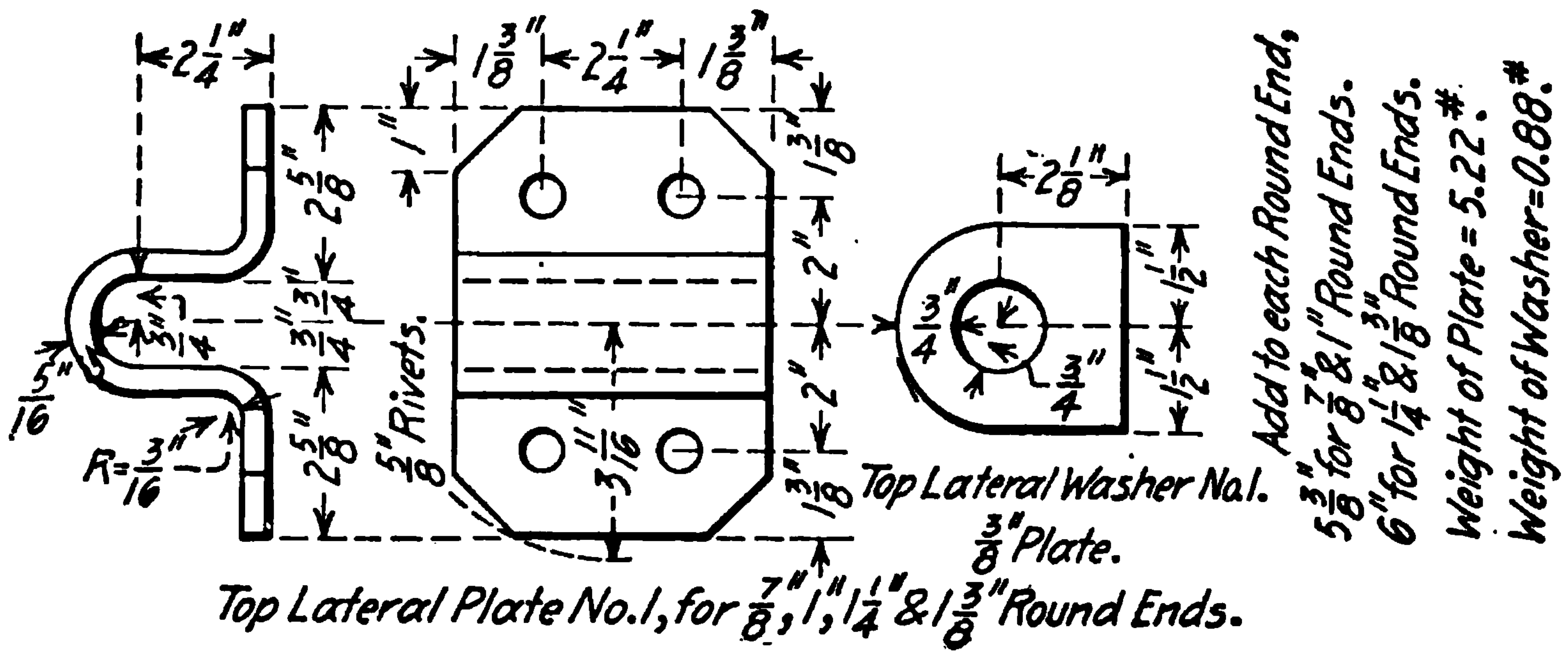


FIG. 207.

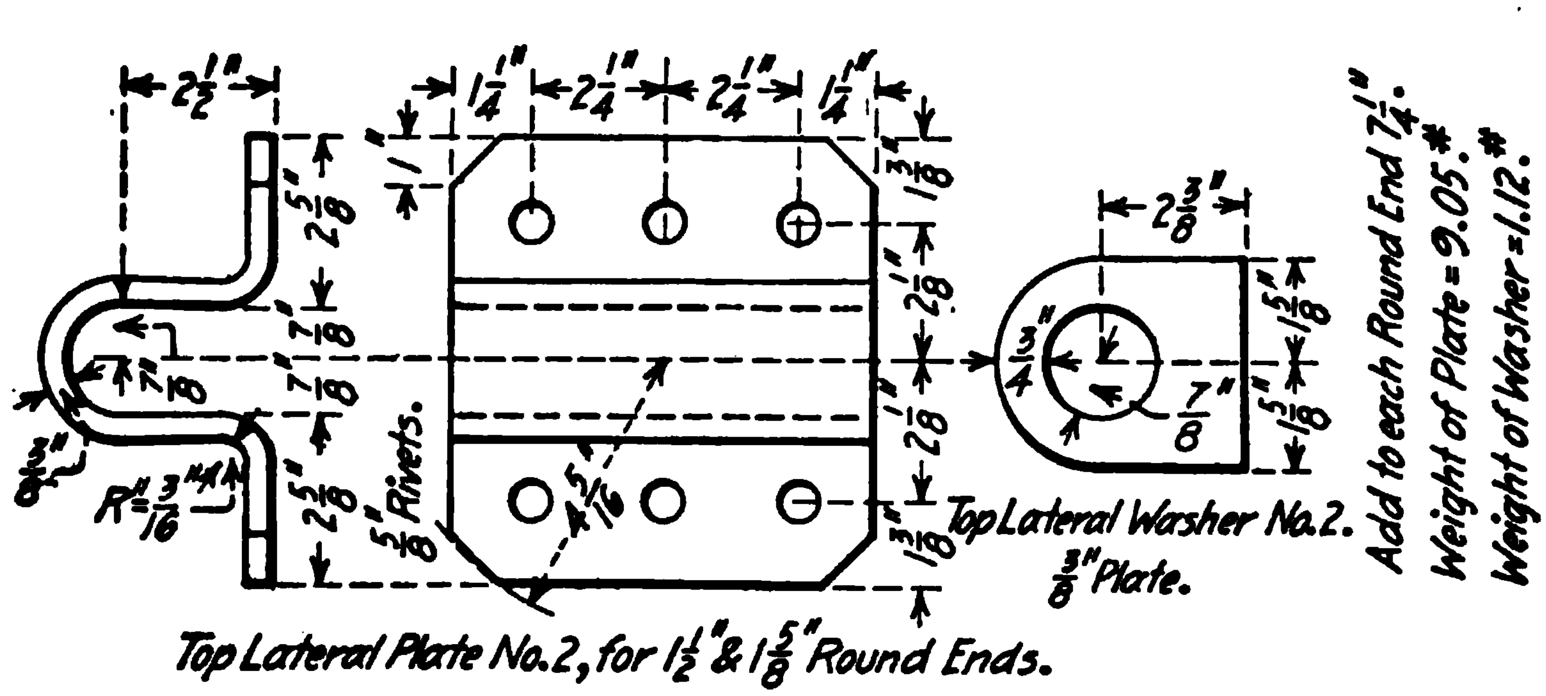


FIG. 208.

detail is quite generally used by bridge companies. Lateral connections may be made by the use of three angles as shown on the 111′ 6″ steel riveted highway bridge over the Illinois and Mississippi Canal in Fig. 163. Lateral connections may also be made by bending a short piece of channel as in Fig. 285. Loop-bars and clevises are used as shown in Fig. 199.

Lower Lateral Connections.—The King Bridge Company's standard lower lateral connections are given in Tables XXXIX, XL and XLI. The connections in Table XXXIX are intended for intermediate

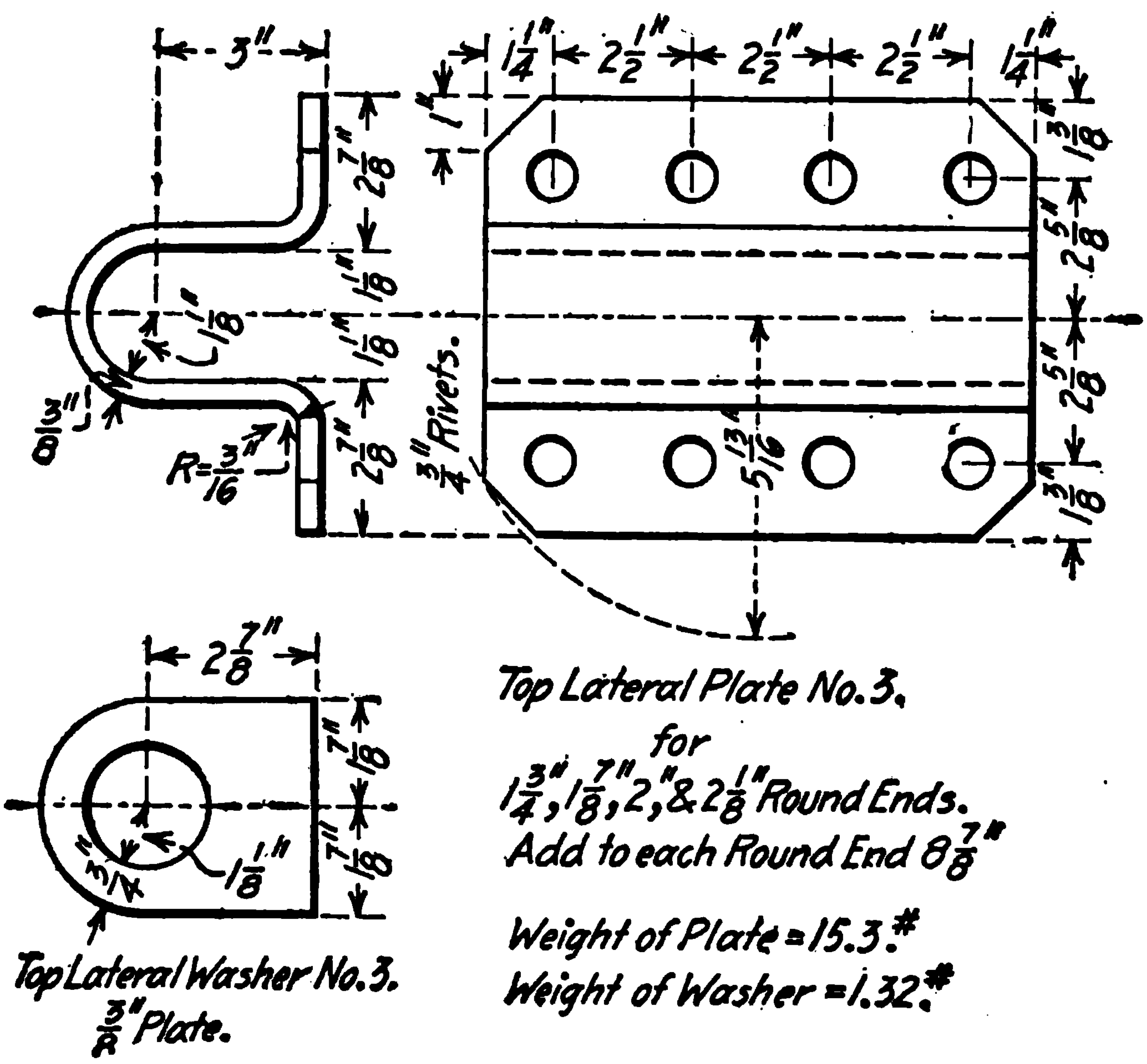
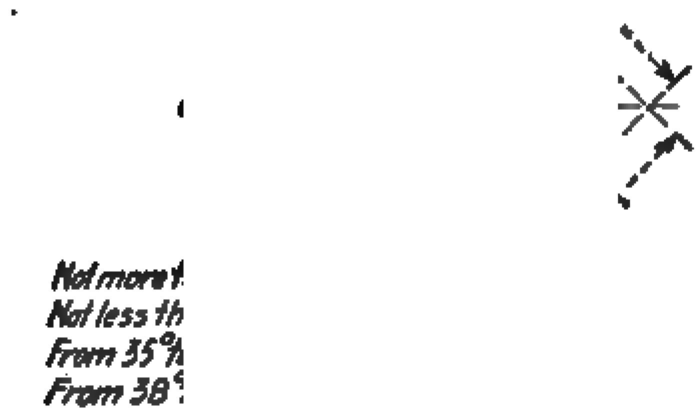


FIG. 209.

floorbeams, while the connections in Table XL are for end floorbeams. Similar connections are used by the American Bridge Company as shown in Figs. 157 and 199, and with clevis connections as shown in Fig. 199. Rods with bent loops are used in Fig. 167; this, however, is very poor practice. The weights of washers to be used with King Bridge Company lateral connections are given in Table XLII.

TABLE XXXIX.

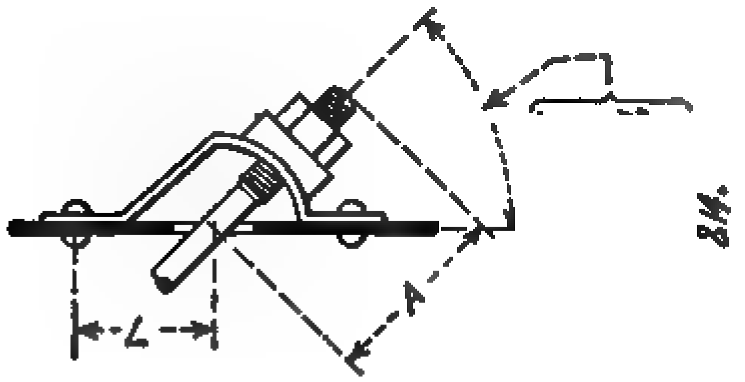
Lower Lateral Connections, King Bridge Co.



Size of Upset End, Ins.	Length A, Ins.	No. of Washer.	No. of Lateral Plate.	No. of Rivets.	Weight of Two Plates, Pounds.
⅞, 1, 1⅛, 1¼	7½	1	1	4	15
1⅜, 1½	7¾	2	2	5	17
1⅝, 1¾	8	3	3	10	25
1⅞, 2, 2⅛	8¼	4	4	12	33
2¼, 2⅜	8½	5	5	12	37
2½, 2⅝	9½	6	6	16	64
2¾, 2⅞, 3	9¾	7	7	20	70

TABLE XL.

Lower Lateral Connections, King Bridge Co.

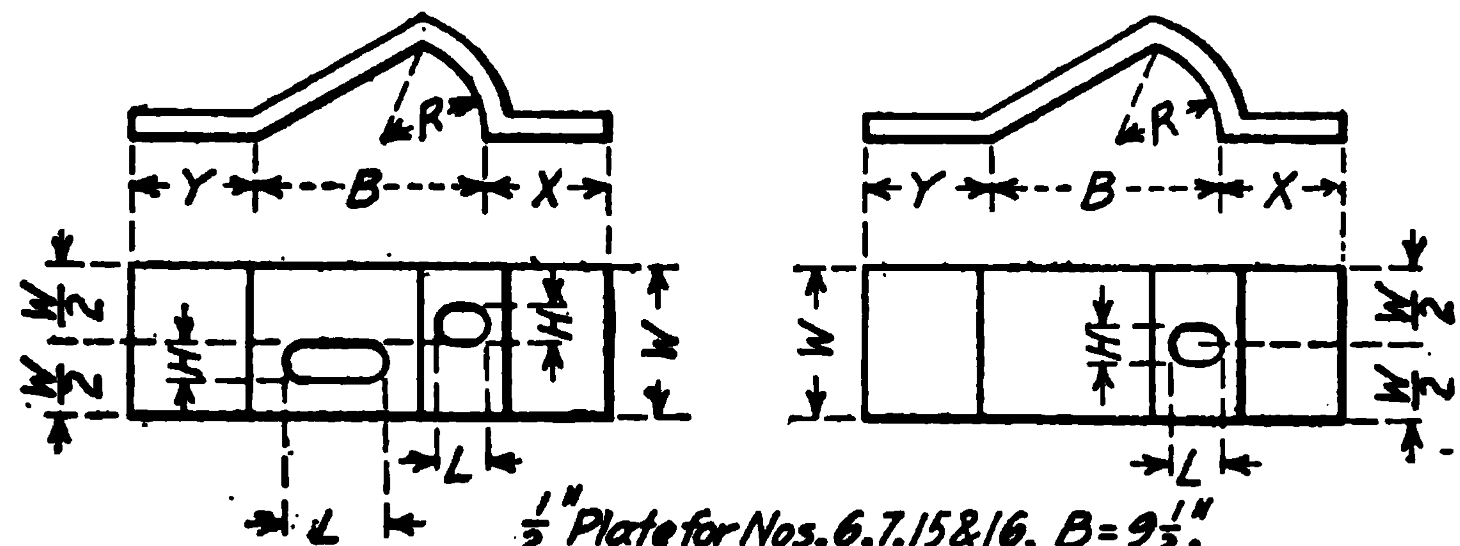


Size of Upset End. Ins.	Length A. Ins.	No. of Washer.	No. of Lateral Plate.	No. of Rivets.	Weight of One Plate. Pounds.
7/8, 1, 1 1/8, 1 1/4	7 1/2	1	10	4	8
1 3/8, 1 1/2	7 3/4	2	11	5	9
1 5/8, 1 3/4	[illegible]	3	12	10	13
1 7/8, 2, 2 1/8	8 1/4	4	13	12	16
2 1/4, 2 3/8	8 1/2	5	14	12	18
2 1/2, 2 5/8	9 1/2	6	15	16	32
2 3/4, 2 7/8, 3	9 3/4	7	16	20	33

TABLE XLI.

Lower Lateral Connections, King Bridge Co.

Dimensions in Inches.

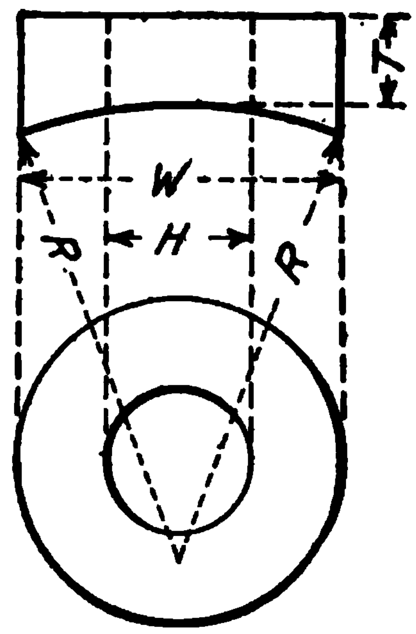


½" Plate for Nos. 6, 7, 15 & 16. B = 9½".
⅜" Plate for Nos. 1, 2, 3, 4, 5, 10, 11, 12, 13 & 14. B = 7½".

Lateral Plate No.	Y	X	H	W	R	L
1	2¼	3½	1½	4½	3⅜	3½
2	2¼	5	1¾	5	3⅜	3½
3	4⅛	5⅞	2	6	3⅜	3½
4	4⅞	6⅛	2⅜	7	3⅜	3½
5	4⅝	6⅜	2⅝	8	3⅜	3½
6	5½	5¾	2⅞	10	4⅜	5
7	8	5¾	3⅛	10½	4⅜	5
10	2¼	3½	1½	4	3⅜	3½
11	2¼	5	1¾	5	3⅜	3½
12	4⅛	5⅞	2	6	3⅜	3½
13	4⅞	6⅛	2⅜	7	3⅜	3½
14	4⅝	6⅜	2⅝	8	3⅜	3½
15	5½	5¾	2⅞	10	4⅜	5
16	8	5¾	3⅛	10	4⅜	5

TABLE XLII.

Cast Washers for Lower Lateral Connections.



No. of Washer.	T Ins.	W Ins.	H Ins.	R Ins.	Weight. Pounds.
1	¾	2¼	1⅜	4	1.0
2	⅞	3¼	1⅝	4	1.6
3	1	3¾	1⅞	4	2.3
4	1	4	2¼	4	2.4
5	1	4½	2½	4	3.1
6	1	4¾	2¾	5	3.3
7	1	5¼	3⅛	5	4.0

CHAPTER XV.

THE DESIGN OF ABUTMENTS AND PIERS.

Introduction.—An abutment is a structure, commonly made of masonry, that supports the ends of the bridge and acts as a retaining wall to hold back the earth fill. A pier is a structure that supports the ends of the bridge where there is no filling to support.

ABUTMENTS.—Abutments are made (1) with a straight main section without wings, (2) with wings making an angle of from 30 to 45 degrees with the axis of the stream, (3) with a T-section, the stem of the T running back into the fill, and (4) with a U-section, where the wings make an angle of 90 degrees with the axis of the stream. Highway bridge abutments are usually made with wing walls, the wings making an angle of 30 to 45 degrees with the face of the abutment. The abutment with wing walls holds the filling and gives a freer channel than any of the other types.

The main part of the abutment should be designed to carry the load and to take the thrust of the earth filling. A common rule is to make the minimum thickness of the main wall not less than 0.4 the height, and then project the footing course 6″ to 8″ on all sides. The bearing of the masonry on the foundation should not be greater than the values given in Table XLIV. Careful tests should always be made and the values given should only be used as an aid to the judgment. The specific gravity, weight and crushing strength of masonry are given in Table XLIII. A factor of safety of from 6 to 10 should be used.

The pressure of the bearing plates on the bridge seats should not exceed 600 lbs. per sq. in. for granite masonry and Portland cement concrete, and 400 lbs. per sq. in. for sandstone and limestone.

Stability of Bridge Abutments Without Wings.—A bridge abutment must be stable (1) against overturning, (2) against sliding, and

TABLE XLIII.

WEIGHT, SPECIFIC GRAVITY AND CRUSHING STRENGTH OF MASONRY.

Materials.	Weight in Pounds per Cubic Foot.	Specific Gravity.	Crushing Strength in Pounds per Square Inch
Sandstone	150	2.4	4,000 to 15,000
Limestone	160	2.6	6,000 to 20,000
Trap	180	2.9	19,000 to 33,000
Marble	165	2.7	8,000 to 20,000
Granite	165	2.7	8,000 to 20,000
Paving brick, Portland cement	150	2.4	2,000 to 6,000
Stone concrete, Portland cement	140 to 150	2.2 to 2.4	2,500 to 4,000
Cinder concrete, Portland cement	112	1.8	1,000 to 2,500

TABLE XLIV.

ALLOWABLE BEARING ON FOUNDATIONS.

Material.	Tons per sq. ft.
Soft clay	1
Ordinary clay and dry sand mixed with clay	2
Dry sand and dry clay	3
Hard clay and firm coarse sand	4
Firm, coarse sand and gravel	6

(3) against crushing the masonry of the foundation or the bridge seat.

1. *Overturning.*—In Fig. 211 let P, represented by OP', be the resultant pressure of the earth, and W, represented by OW, be the weight of the abutment and load on the abutment corrected for buoyancy of water under the pier acting through its center of gravity. Then E, represented by OR, will be the resultant pressure tending to overturn the abutment.

Draw OS through the point A. For this condition the abutment should be on the point of overturning, and the factor of safety against overturning would be unity. The factor of safety for $E = OR$ will be

$$f_0 = SW/RW \tag{94}$$

2. *Sliding.*—In Fig. 211 construct the angle $H1G$ equal to ϕ', the angle of friction of the masonry on the foundation. Now if E passes through 1, and takes the direction OQ, the abutment will be on the point of sliding, and the factor of safety against sliding, f_s, will be unity. For $E = OR$, the factor of safety against sliding will be

$$f_s = QM'/RM \tag{95}$$

If the resultant at 3 be resolved into a vertical force F, and a horizontal force H (not shown in Fig. 211), then

$$f_s = F/(H \cdot \tan\phi') \tag{96}$$

which will give the same result as above.

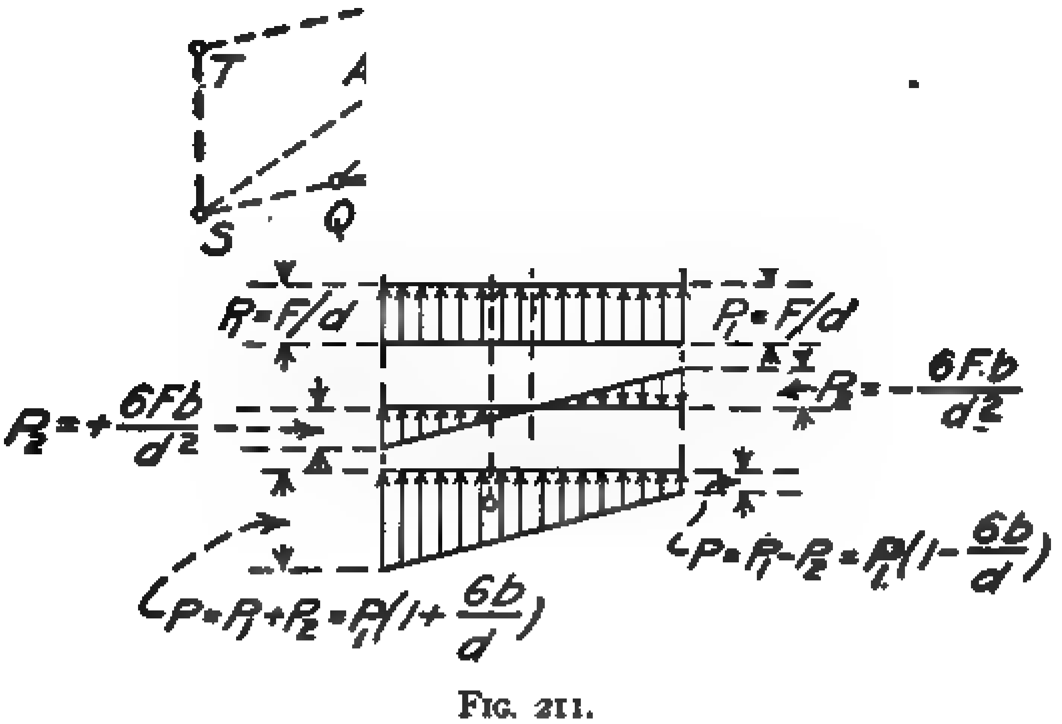


FIG. 211.

Bridge abutments seldom fail by sliding unless water has undermined the structure.

3. *Crushing.*—In Fig. 211 the load on the foundation will be due to a vertical force F, which produces a uniform stress $p_1 = F/d$ over the area of the base, and a bending moment $= F \cdot b$, which produces compression, p_2, on the front and tension, p_2, on the back of the foundation. The sum of the tensile stresses due to bending must equal the sum of the compressive stresses, $= \frac{1}{4}p_2 \cdot d$. These stresses act as a couple

through the centers of gravity of the stress triangles on each side, and the resisting moment is

$$M' = \tfrac{1}{4}p_2 \cdot d \cdot \tfrac{2}{3}d = \tfrac{1}{6}p_2 \cdot d^2 \qquad (97)$$

But the resisting moment equals the overturning moment, and

$$\tfrac{1}{6}p_2 \cdot d^2 = F \cdot b,$$

and

$$p_2 = \pm\, 6F \cdot b/d^2 \qquad (98)$$

The total stress on the foundation then is

$$p = p_1 \pm p_2 = p_1(1 \pm 6b/d) \qquad (99)$$

Now if $b = \tfrac{1}{6}d$, we will have

$$p = 2p_1, \text{ or } 0.$$

In order therefore that there be no tension, or that the compression never exceed twice the average stress the resultant should never strike outside the middle third of the base.

If the resultant strike outside of the middle third of a bridge abutment in which the masonry can take no tension, the load will all be taken by compression and can be calculated as follows:

In Fig. 212 the resultant F will pass through the center of gravity of the stress diagram, and will equal the area of the diagram.

$$F = {}^3\!/_2 p \cdot a,$$

and

$$p = \tfrac{2}{3}F/a \qquad (100)$$

which gives a larger value of p than would be given if the masonry could take tension.

For the design of retaining walls, see the author's "The Design of Walls, Bins and Grain Elevators."

Design of Bridge Piers.—Bridge piers must be designed (1) for the total vertical load due to the dead and live load of the span and the weight of the pier; (2) for wind pressure on the pier and the bridge; (3) to withstand floating drift and ice; and (4) to take the

longitudinal thrust due to stopping a car or train on the bridge, and due to temperature when the rollers do not move freely. The wind pressures are calculated as specified in Chapter II and Chapter VII, and are assumed to act in the vertical line of the center of the pier; on the top chord of the truss; the bottom chord of the truss; 6 feet

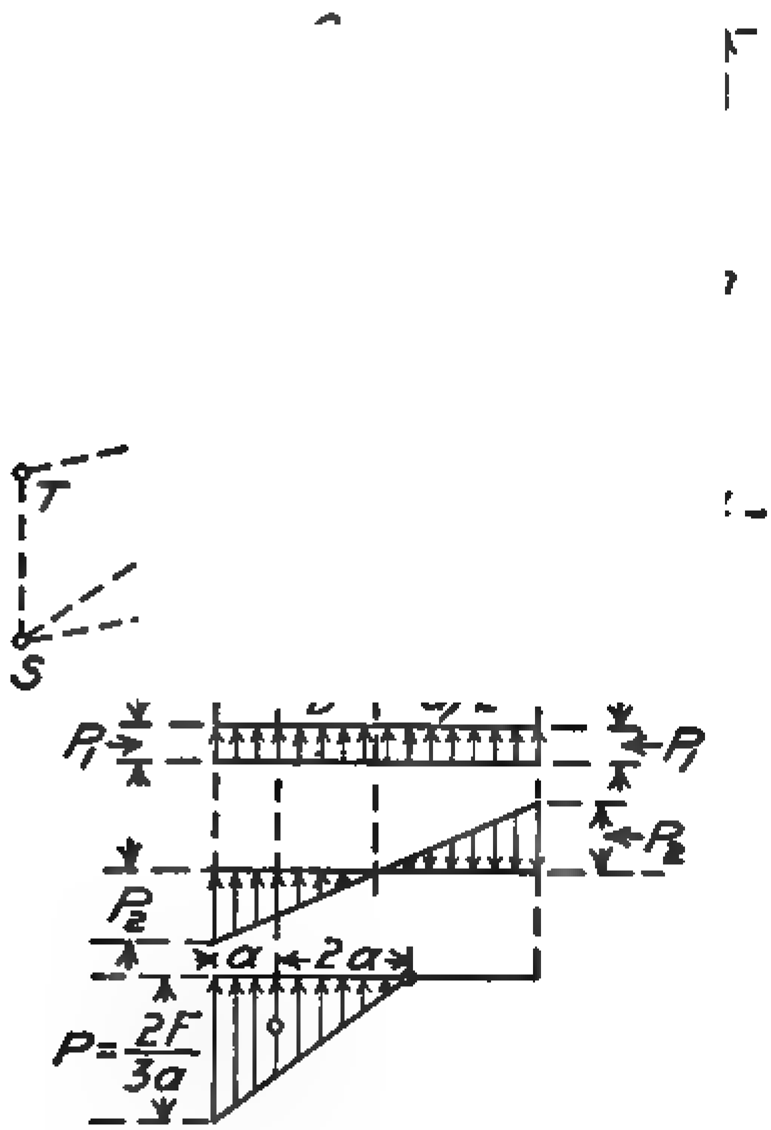


FIG. 212.

above the base of the rail; and at the center of gravity of the exposed part of the pier. The total wind moment is then calculated about the leeward edge of the base of the pier, and the maximum stresses on the foundation due to direct load and wind are calculated in the same manner as the calculation of the pressures of abutments.

The effect of the current of the stream and of floating ice and drift are difficult to calculate. The maximum pressure due to floating ice will be the crushing strength of the ice, which varies from 400 to 800

lbs. per square inch. The principal danger from floating ice and drift is that the current of the stream will be deflected downward and will gouge out the material around and under the pier and cause failure. To prevent this it is quite common to build piers with a "break-water" or nose that will deflect drift and ice, or to put in a pile protection on the upstream side of the pier. If the water can get under the pier the buoyancy of the water must be considered in calculating the safety. If there is danger of scouring then it is well to deposit large stones and riprap around the base of the pier.

Batter.—Piers and abutments are seldom battered more than one inch to one foot of vertical height, or less than one-half inch to the foot, although high piers are sometimes battered only one-fourth inch to one foot.

Preparing the Foundations.—The preparation of the site of the abutment or pier will depend upon the conditions and character of the material.

Rock.—Where the water can be excluded, the rock should be cleared of all overlying material and disintegrated rock. The surface is then leveled up either by cutting off the projections or by depositing a layer of concrete.

Hard Ground.—The material should be excavated well below the frost and scour line. Where the foundations cannot be carried low enough to prevent undermining, piles should be driven at about 2½ to 3 feet centers over the foundation.

Soft Ground.—The materials should be excavated to a solid stratum or piles spaced about 2½ to 3 feet centers should be driven over the foundation to a good refusal. The piles should be cut off below low water level to carry a timber grillage, or concrete may be deposited around the heads of the piles. Where the water cannot be excluded it will be necessary to use one of the following methods: open caisson, crib, coffer dam, or pneumatic caisson.

In using an open caisson the masonry is built up or the concrete is deposited in a water tight box built of heavy timbers, the caisson being sunk as the pier is built up. The caisson is commonly floated into

place and then is sunk on piles which have been sawed off to receive it, or on a solid rock foundation. The sides of the caisson are usually removed after the pier is completed.

Timber cribs are made of squared timbers placed transversely and longitudinally, and bolted together so as to form a solid structure with open pockets. The crib is sunk by loading the pockets with stone. No timber should be left above the low water mark in open caissons or cribs.

A coffer dam is usually made by driving two rows of sheet piling around the pier, the space between the rows of piling being filled with clay puddle. For small depths a single row of sheet piling is often sufficient. Where the depth is too great for one length of sheet piling, additional rows are driven inside the first. Steel sheet piling is now much used for difficult foundations. Steel sheet piling can be driven through ordinary drift and similar material, is not limited in depth, and is practically water tight when used in a single row. It can be drawn and used again. It is almost impossible to shut off all the water with a coffer dam, and pumps should be provided.

Pneumatic caissons should only be used under the direction of experienced engineers and will not be considered here.

Timber and Piling.—All timber and piles should be of sound white oak, pine or other timber equally good, and should conform to the specifications in Chapter XVIII. Piles should be driven 8 to 15 feet in gravel, sand or stiff clay, and 20 to 30 feet in soft clay, and should not give a penetration of more than one inch for each of the last six blows with a 2,000-lb. hammer dropping freely 20 ft.

The safe bearing load, where gravity pile drivers are used, should be calculated by means of the Engineering News' formula:

$$P = 2W \cdot h/(s + 1) \tag{101}$$

where P = the safe load on the pile in tons;

W = the weight of the hammer in tons;

s = the penetration of the pile for the last blow in inches;

h = the free fall of the hammer in feet.

Where steam hammers are used the coefficient in the denominator should be 0.1 in place of 1. Care should be used not to overdrive piles.

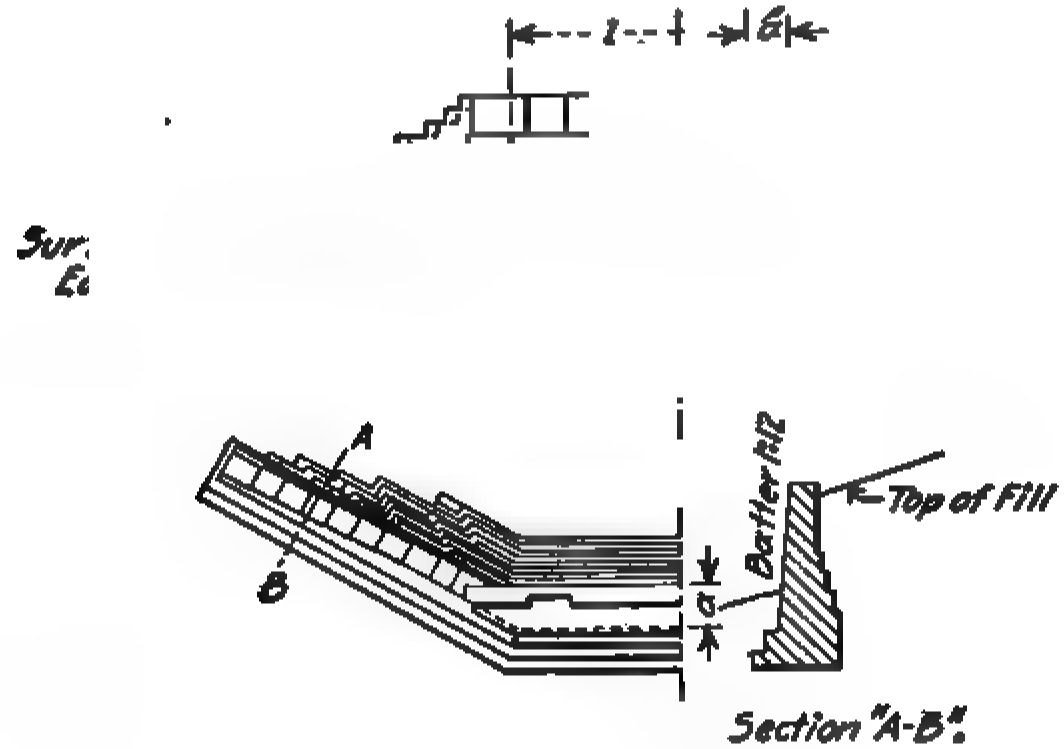


FIG. 213. COOPER'S STANDARD MASONRY ABUTMENTS WITH WING WALLS FOR HIGHWAY BRIDGES.

Cooper's Standard Abutments.—The abutment in Fig. 213 is from Cooper's "General Specifications for Foundations and Substructures of Highway and Electric Railway Bridges." The length, *l*, and the thickness, *a*, for highway and single track electric railway bridges are as given in Table XLV, and proportional for intermediate spans. These abutments may be made of either first-class stone masonry, or first-class Portland cement concrete.

For double track electric railway bridges add one foot to the value of (a) in Table XLV. The minimum thickness of the wall at any

TABLE XLV.

DIMENSIONS OF MASONRY ABUTMENTS IN FIG. 213, COOPER'S STANDARDS.

DISTANCE, *a*.	SPAN IN FEET.	LENGTH, *l*.
2′ 6″	50	*w* + 4′ 0″
2 8	100	*w* + 5 0
3 0	150	*w* + 5 9
3 4	200	*w* + 6 6
3 6	250	*w* + 7 0

point to be 0.4 of the height. The length of the wing walls will be determined by local conditions.

The abutment without wing walls in Fig. 214 has the same dimensions as given in Table XLV. The width of single track electric railways may be taken as 14 feet, double track 26 feet. The approximate

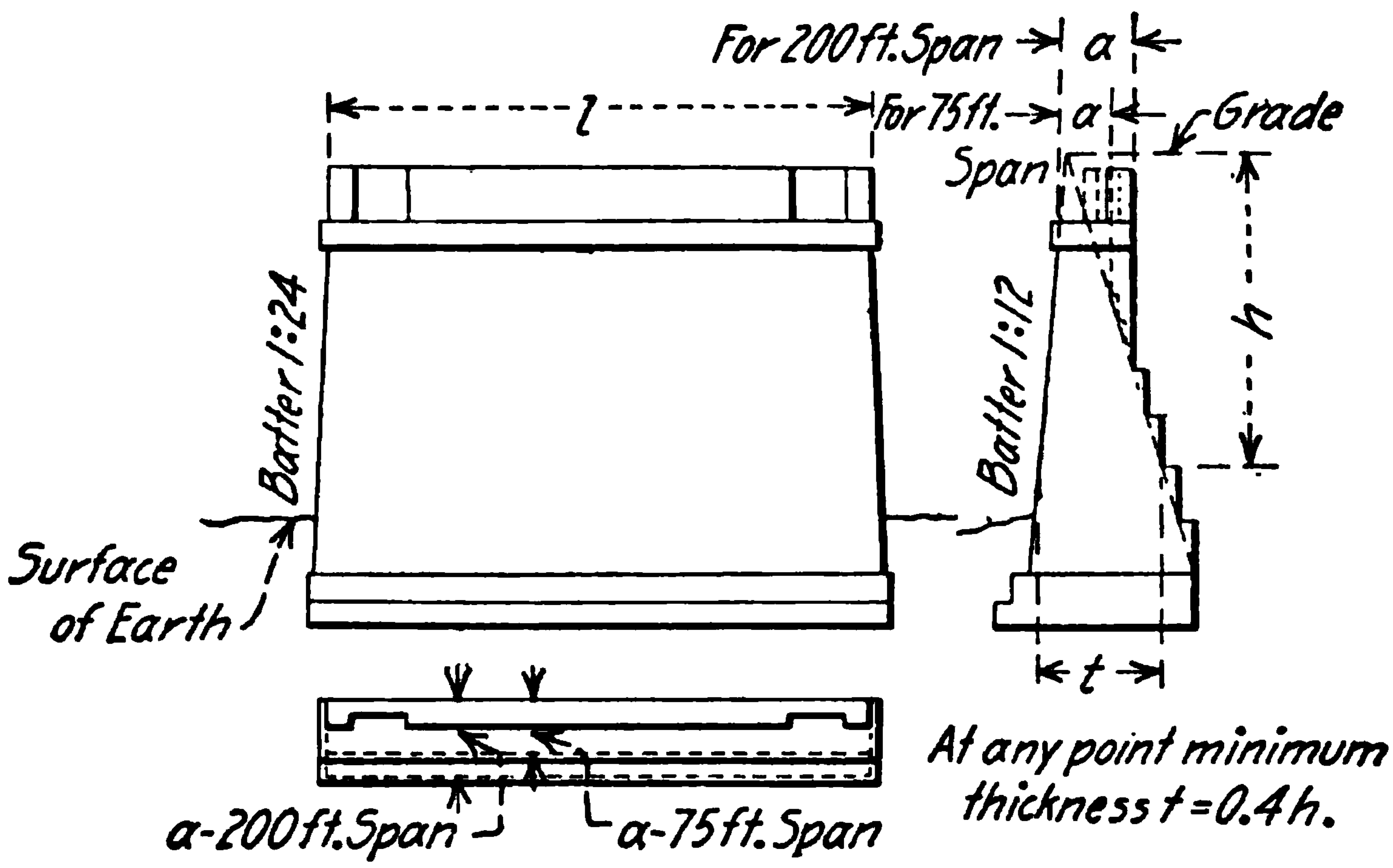


FIG. 214. COOPER'S STANDARD MASONRY ABUTMENTS WITHOUT WING WALLS FOR HIGHWAY BRIDGES.

cubical quantities in abutments without wing walls are given in Table XLVI.

TABLE XLVI.

APPROXIMATE CONTENTS IN CUBIC YARDS OF ONE MASONRY ABUTMENT, WITHOUT WING WALLS. FIG. 214.

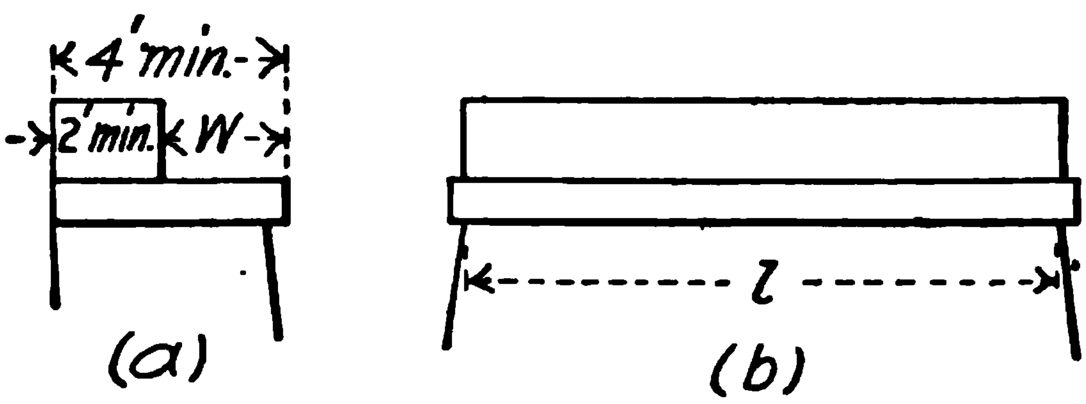
Span. Feet.	Roadway.	Depth of Footing Below Grade, Feet. 10	15	20	25	30
100	12 feet	20	39	67	100	145
	20 feet	28	56	95	145	206
	E, single T	21	44	75	112	160
	E, double T	36	72	120	183	260
300	12 feet	22	45	77	116	165
	20 feet	31	63	106	161	227
	E, single T	25	50	85	128	181
	E, double T	49	84	141	210	296

Schneider's Standard Abutments.—Mr. C. C. Schneider gives the standards in Fig. 215 and Table XLVII for abutments for electric railway bridges. The dimensions refer to the three classes which are described in Chapter II:

Class "A," loading for heavy traffic;

Class "B," loading for medium traffic;

Class "C," loading for light traffic.



(*a*) (*b*)

FIG. 215. MASONRY ABUTMENTS FOR ELECTRIC RAILWAY BRIDGES, SCHNEIDER'S STANDARDS.

The sizes given in Table XLVII are minimum dimensions for abutments. The masonry coping is to project at least 3 inches beyond the body under the coping on the sides. The thickness of abutments under

TABLE XLVII.

DIMENSIONS OF MASONRY ABUTMENTS IN FIG. 215, SCHNEIDER'S STANDARDS.

Span of Bridge. Feet.	Thickness w in Feet.						Length l = Distance c to c Trusses + Figures Below.					
	Class A.		Class B.		Class C.		Class A.		Class B.		Class C.	
	S. T.	D. T.	S. T.	D. T.	S. T.	D. T.	S. T.	D. T.	S. T.	D. T.	S. T.	D. T.
	' "	' "	' "	' "	' "	' "	' "	' "	' "	' "	' "	' "
50	2-2	2-9	2-0	2-2	2- 0	2- 0	3-6	4-0	3-6	3-6	3-6	3-6
100	2-8	3-6	2-0	2-8	2- 0	2- 2	4-0	5-0	3-6	4-0	3-6	3-6
150	3-0	4-0	2-4	3-0	2- 0	2- 6	4-6	5-6	4-0	4-6	3-6	4-0
200	3-4	4-6	2-8	3-4	2- 2	2-10	5-0	6-0	4-0	5-0	3-6	4-6
300	4-0	5-6	3-1	4-0	2- 7	3- 6	5-6	7-0	4-6	5-6	4-0	5-0
400	4-8	6-2	3-5	4-8	2-11	4- 0	6-0	7-6	5-0	6-0	4-6	5-6

the coping shall in no case be under 4 feet, and the thickness of the parapet wall back of abutments not less than 2 feet. These abutments may be made of either first-class stone masonry, or first-class Portland cement concrete.

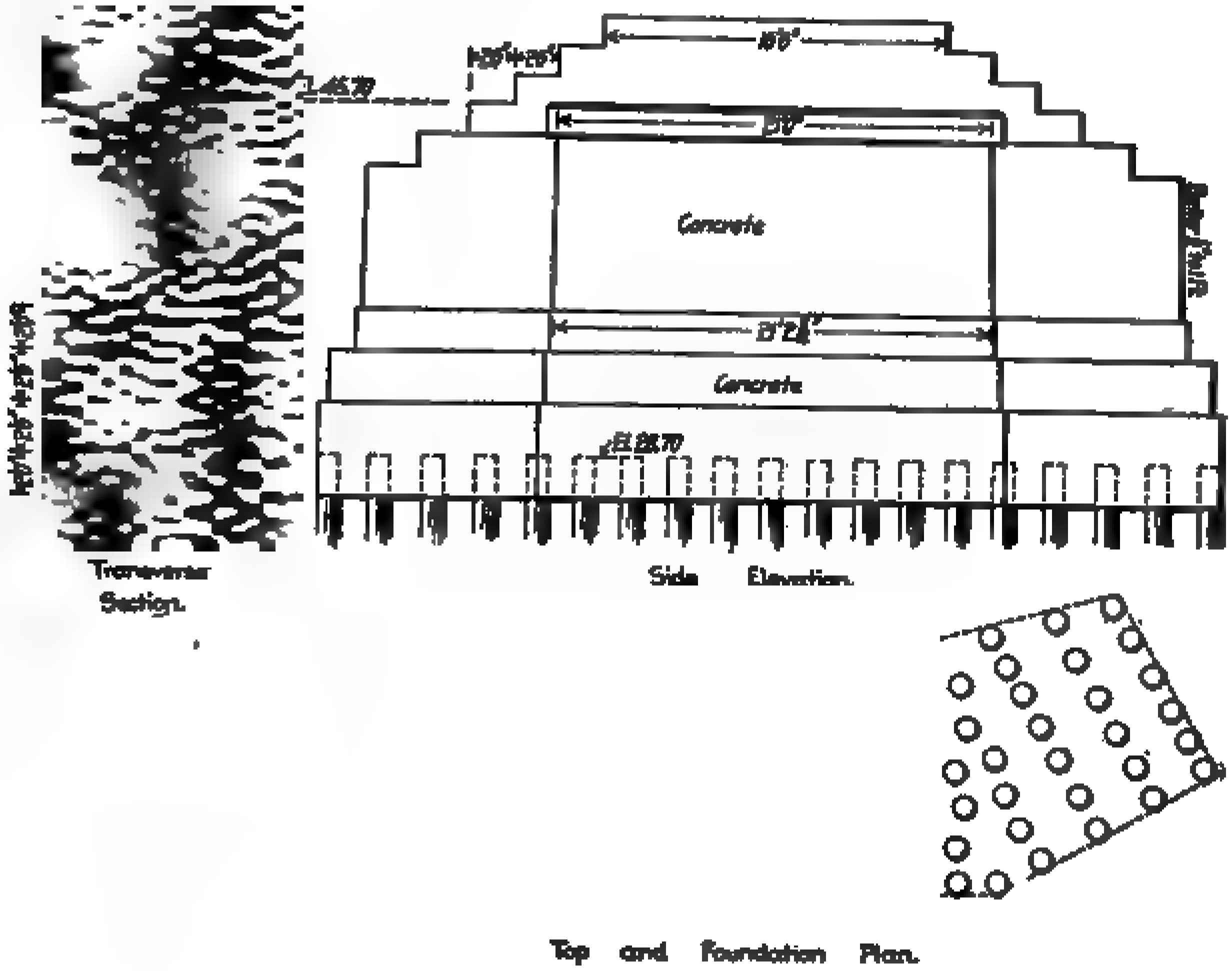


FIG. 216. CONCRETE ABUTMENT FOR 156-FT. SPAN ELECTRIC RAILWAY BRIDGE OVER KICKAPOO CREEK, PEORIA & PEKIN TERMINAL R. R.

FIG. 217. CONCRETE ABUTMENTS FOR 111′ 6″ SPAN STEEL RIVETED HIGHWAY BRIDGE ACROSS ILLINOIS AND MISSISSIPPI CANAL.

The concrete abutment shown in Fig. 216 was designed to carry a 156-ft. span electric railway bridge. This abutment is carried on timber piles driven to a firm bearing.

The concrete abutments for a 111′ 6″ riveted highway bridge across the Illinois and Mississippi Canal are shown in Fig. 217.

Reinforced Concrete Abutments.—Reinforced concrete abutments for a single track plate girder bridge, as designed by the C. B. & Q. R. R., are shown in Fig. 218. The details are shown in the cut and need no further explanation.

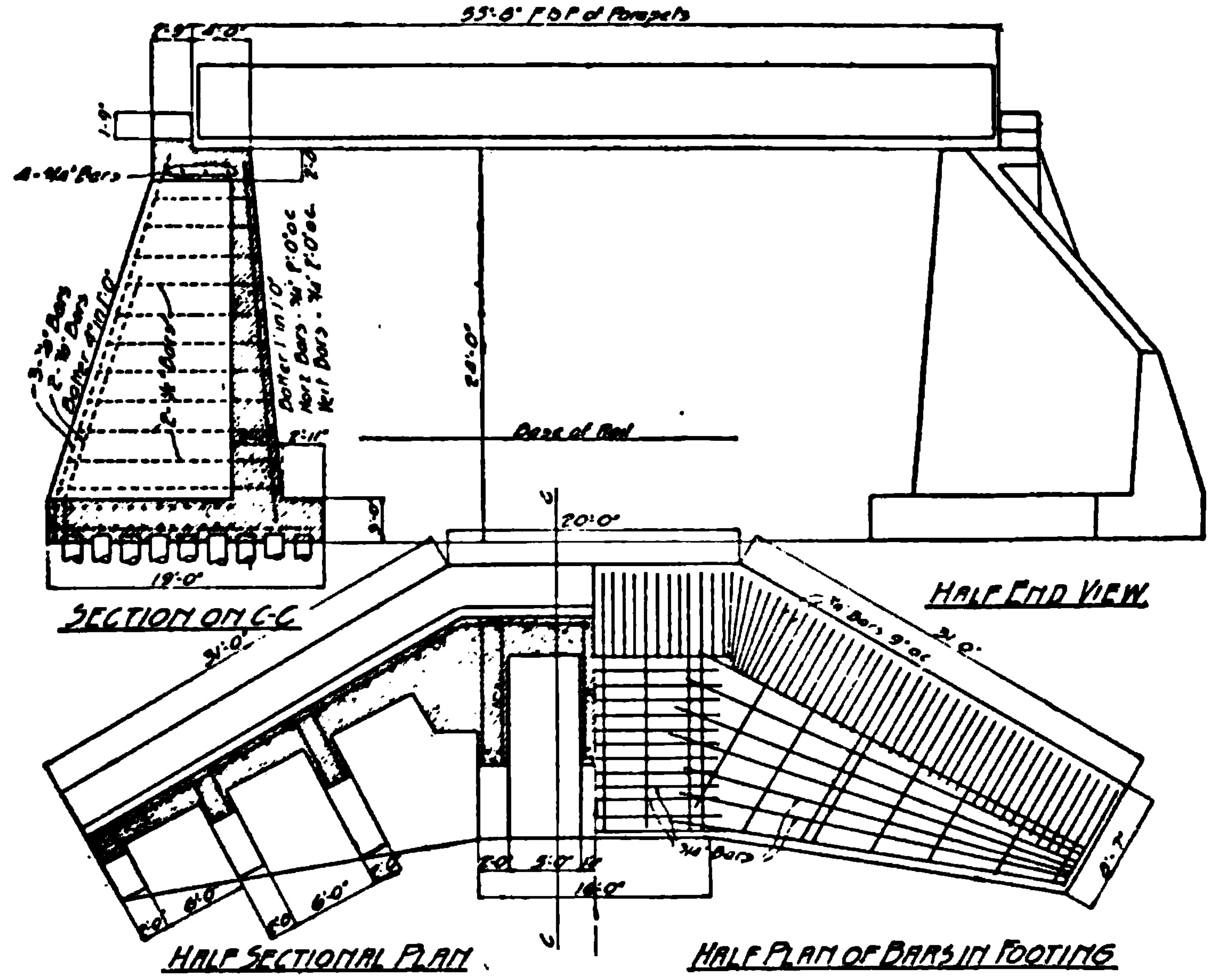


FIG. 218. REINFORCED CONCRETE RAILWAY BRIDGE ABUTMENT, C. B. & Q. R. R.

The standard reinforced concrete abutments for highway bridges, designed by the U. S. Reclamation Service for the Garland Canal, Shoshone Project, are shown in Fig. 219, while the quantities and dimensions are given in Table XLVIII.

Cooper's Standard Masonry Piers.—The masonry pier in Fig. 220 is from Cooper's "General Specifications for Foundations and Sub-

TABLE XLVIII.

VARIABLE DIMENSIONS AND QUANTITIES FOR HIGHWAY BRIDGE ABUTMENTS WITH WING WALLS, FIG. 219.

h = Height of Abutment in Feet.	Type of Truss.	Type of End Shoe.	f = Bottom Width of Pier Buttress.	g = Total Width of Heel Slab in Pier.	k = Distance from Center to Break in Heel.	l = Length of Wing Wall.	Quantities for One Abutment.						
							Cubic Yards of Concrete.			Pounds of Plain Steel Bars.			
							Piers.	Wing Walls.	Total.	½″ Square.	¾″ Square.	1″ Square.	Total.
1	2	3	4	5	6	7	8	9	10	11	12	13	14
6	Pony	Both Shoes	3′-6″	5′-3″	14′- 2″	6′- 6″	10.0	2.7	12.7	400	570	160	1,130
	Thru	Roller	4-0	5-9	16- 6	6- 0	12.4	3.0	15.4	410	640	160	1,210
		Fixed			16- 3	6- 6		2.6	15.0				
8	Pony	Both Shoes	4-0	5-9	13- 8	10- 0	12.8	4.9	17.7	530	790	260	1,580
	Thru	Roller	4-6	6-3	15-11	9- 6	16.1	4.9	23.5	540	840	260	1,640
		Fixed			15- 8	10- 0		5.0	23.6				
10	Pony	Both Shoes	4-6	6-3	13- 2	13- 6	15.7	7.8	23.5	660	1,050	330	2,040
	Thru	Roller	5-0	6-9	15- 4	13- 0	20.0	7.6	27.6	670	1,090	330	2,090
		Fixed			15- 1	13- 6		8.1	28.1				
12	Pony	Both Shoes	5-0	6-9	12- 6	17- 0	18.9	11.7	30.6	810	1,500	720	3,030
	Thru	Roller	5-6	7-3	14- 9	16- 6	24.1	11.3	35.4	820	1,540	720	3,080
		Fixed			14- 6	17- 0		11.8	35.9				
14	Pony	Both Shoes	5-6	7-3	12- 0	20- 5	22.2	16.3	38.5	965	1,860	860	3,685
	Thru	Roller	6-0	7-9	14- 3	19-11	28.5	15.8	44.3	970	1,910	860	3,740
		Fixed			14- 0	20- 5		16.4	44.9				
16	Pony	Both Shoes	5-9	7-9	11- 6	23-10	27.4	21.8	49.2	1,130	2,940	1,520	5,590
	Thru	Roller	6-3	8-3	13- 6	23- 5	34.4	21.4	55.8	1,140	3,030	1,520	5,690
		Fixed			13- 5	23-10		23.0	57.4				
18	Pony	Both Shoes	6-3	8-3	10-11	27- 4	31.3	28.4	59.7	1,300	3,520	1,780	6,680
	Thru	Roller	6-9	8-9	13- 2	26-10	40.0	27.5	67.5	1,310	3,560	1,780	6,650
		Fixed			12-11	27- 4		28.3	68.3				
20	Pony	Both Shoes	6-9	8-9	10- 9	30-10	35.4	35.7	71.1	1,470	4,150	2,160	7,780
	Thru	Roller	7-3	9-3	12- 6	30- 4	45.0	35.0	80.0	1,480	4,260	2,160	7,900
		Fixed			12- 3	30-10		36.0	81.0				

NOTES.—1. The abutments for pony type steel trusses are intended for a span of 75 feet or less; those for through type steel trusses for a span of 100 feet or less.

2. The abutments have been designed for standard highway bridges and for an earth pressure based on a weight of 110 pounds per cubic foot, an angle of repose of 26 degrees and a bearing power of soil of 5000 pounds per square foot.

3. The depth of foundations should be sufficient to prevent dangerous scouring or undermining and to provide the required bearing value of soil.

RECLAMATION SERVICE ABUTMENT.

structures of Highway and Electric Railway Bridges." The length, *l*, and the thickness, *a*, for highway and single track electric railway bridges are given in Table XLIX. These piers may be made of either first-class stone masonry, or first-class Portland cement concrete.

TABLE XLIX.

DIMENSIONS OF MASONRY PIER FOR HIGHWAY AND SINGLE TRACK ELECTRIC RAILWAY BRIDGES, AS IN FIG. 220.

DISTANCE, *a*.	SPAN IN FEET.	LENGTH, *l*.
2′ 8″	50	$w+4'$ 0″
2 10	75	$w+4$ 6
3 2	100	$w+5$ 0
3 8	150	$w+5$ 9
4 4	200	$w+6$ 6
4 10	250	$w+7$ 0
5 4	300	$w+7$ 6

For double track electric railway bridges add one foot to *l*, and from 4 to 6 inches to *a*. The width, *b*, must not be less than 0.4 the height. Where drift or logs are liable to injure the pier the nose of the cut-water should be protected with a steel angle or plate.

The approximate cubical contents of the piers shown in Fig. 220 and Table XLIX are given in Table L.

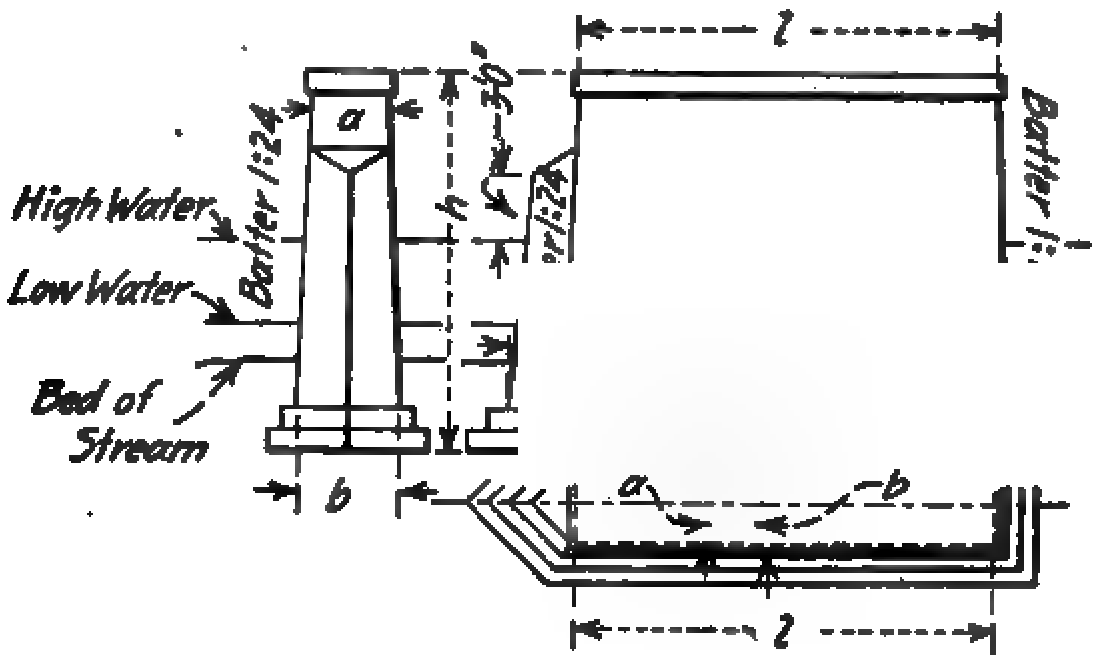


FIG. 220. COOPER'S STANDARD MASONRY PIERS FOR HIGHWAY BRIDGES.

TABLE L.

APPROXIMATE CONTENTS IN CUBIC YARDS OF ONE MASONRY PIER. FIG. 220.

Spans. Feet.	Roadways.	Depth of Pier from Top of Coping to Bottom of Footing in Feet.				
		10	15	20	25	30
100	12 feet	29	44	60	77	94
	20 feet	38	59	82	108	136
	E, single T	31	46	62	80	100
	E, double T	50	75	102	132	166
150	12 feet	34	51	70	90	111
	20 feet	46	70	95	125	157
	E, single T	37	54	74	96	120
	E, double T	58	86	118	153	191
200	12 feet	39	58	80	103	128
	20 feet	53	80	109	143	178
	E, single T	43	63	86	112	140
	E, double T	66	99	135	174	217
250	12 feet	44	66	90	116	145
	20 feet	61	91	123	160	199
	E, single T	48	74	98	127	159
	E, double T	73	109	149	192	238
300	12 feet	49	73	100	130	162
	20 feet	68	101	137	177	220
	E, single T	54	80	109	142	178
	E, double T	80	120	164	210	260

Schneider's Standard Masonry Piers.—Mr. C. C. Schneider's standards for masonry piers for electric railway bridges are given in Fig. 221 and in Table LI. The thickness of the piers was determined on the assumption that the pier supports two spans of approximately the same length. The top of the pier should have sufficient room for the bridge seat; the bed plates should in no case come nearer than 3

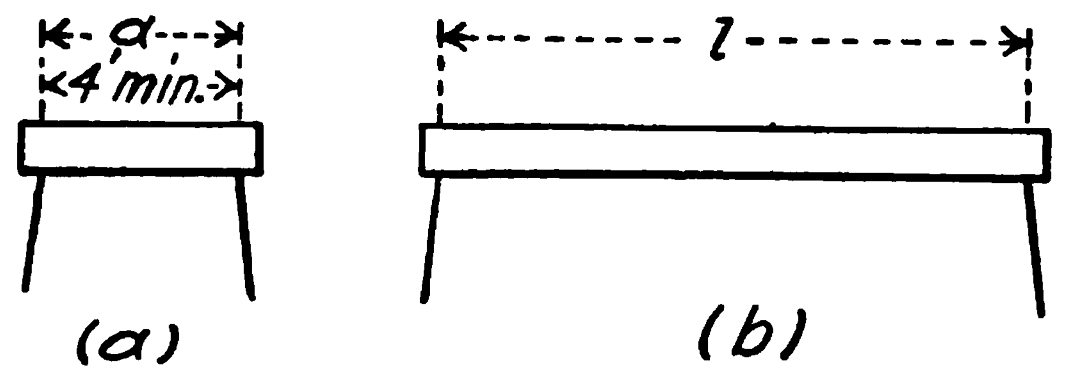


FIG. 221. MASONRY PIERS FOR ELECTRIC RAILWAY BRIDGE, SCHNEIDER'S STANDARDS.

inches to the edge of the masonry under the coping on the side of the pier, nor nearer than 6 inches to the edge of the masonry under the

coping on the end of the pier. These piers may be made of either first-class stone masonry, or first-class Portland cement concrete.

TABLE LI.

DIMENSIONS OF MASONRY PIERS IN FIG. 221, SCHNEIDER'S STANDARDS.

Span of Bridge, Feet.	Thickness "a" Under Coping.						l = Distance c to c of Trusses + Figures in Table.					
	Class A.		Class B.		Class C		Class A.		Class B.		Class C.	
	S. T.	D. T.	S. T.	D. T.	S. T	D. T.	S. T.	D. T.	S. T.	D. T.	S. T	D. T.
	′ ″	′ ″	′ ″	′ ″	′ ″	′ ″	′ ″	′ ″	′ ″	′ ″	′ ″	′ ″
50	4-0	5-3	4-0	4-0	4-0	4-0	3-6	4-0	3-6	3-6	3-6	3-6
100	5-0	6-6	4-6	5-0	4-0	4-0	4-0	5-0	3-6	4-0	3-6	3-6
150	5-8	7-6	4-3	5-8	4-0	4-8	4-6	5-6	4-0	4-6	3-6	4-
200	6-4	8-6	4-9	6-4	4-0	5-4	5-0	6-0	4-0	5-0	3-6	4-60
300	7-8	10-6	5-0	7-8	4-10	6-6	5-6	7-6	4-6	5-6	4-0	5-0
400	9-0	12-0	6-6	9-0	5-6	7-6	6-0	7-6	5-0	6-0	4-6	5-6

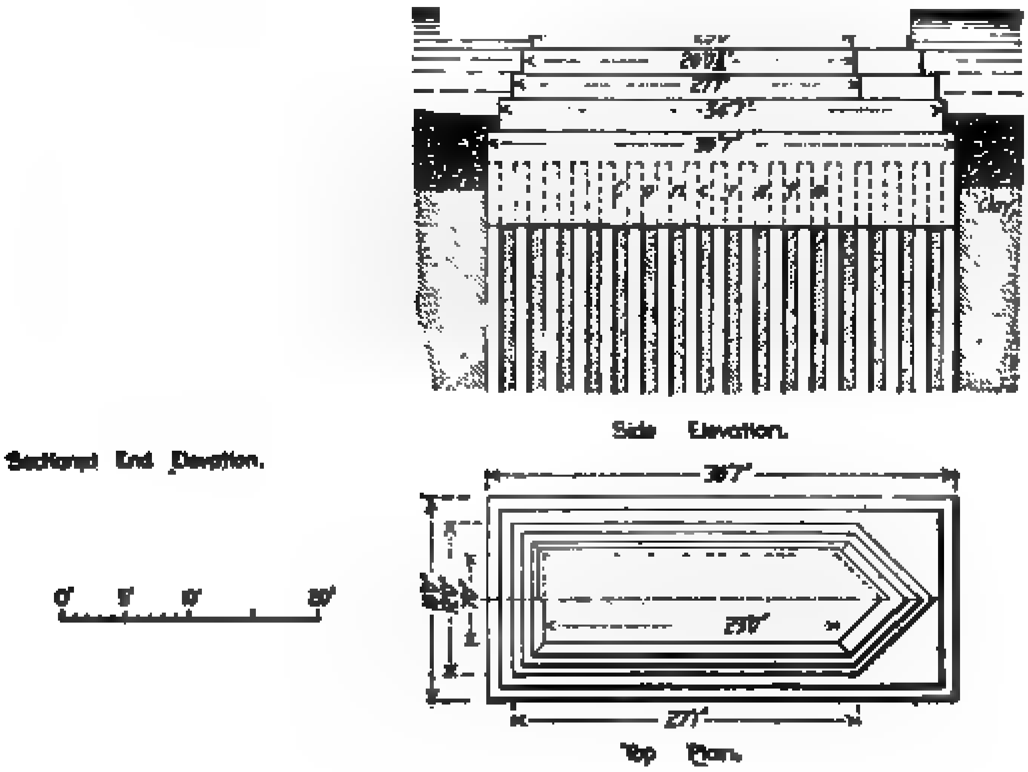


FIG. 222. CONCRETE PIER FOR 156-FT SPAN ELECTRIC RAILWAY BRIDGE OVER ILLINOIS RIVER, PEORIA & PEKIN TERMINAL R. R.

The concrete pier in Fig. 222 was designed to carry two 156-ft. span electric railway bridges. The pier is carried on timber piles driven to a firm bearing. This pier is considerably larger than required by Cooper's specifications. A masonry pivot pier for a swing bridge for the same crossing is given in Fig. 223. The swing bridge has a clear span of 170 feet, and a length of 376 feet.

FIG. 223. PIVOT PIER FOR 376-FT. ELECTRIC RAILWAY SWING SPAN, ILLINOIS RIVER, PEORIA & PEKIN TERMINAL R. R.

SPECIFICATIONS.—The standard specifications adopted by the American Railway Engineering and Maintenance of Way Association are now used on first-class work and are given on the following pages.

DEFINITIONS OF MASONRY TERMS.*

Masonry.—All constructions of stone or kindred substitute materials in which the separate pieces are either placed together, with or without cementing material to join them, or where not separately placed, are encased in a matrix of firmly cementing material.

Riprapping and paving are not masonry construction.

Classification of Masonry.

Kinds of Masonry.	Sub-division into Classes of Material.	Manner of Work.	Dressing.	Face or Surface Finish.
Stone.	Dimension.	In courses.	Fine-pointed. Crandalled. Axed or Pean-hammered. Tooth-axed. Sawed. Rubbed.	Quarry-faced. Pitch-faced. Drafted.
	Arch.	In courses.	Fine-pointed. Crandalled. Axed or Pean-hammered. Tooth-axed. Sawed.	
	Ashlar.	Range. Broken range.		
	Squared stone.	Range. Broken range. Random.	Rough-pointed. Fine-pointed. Crandalled.	
	Rubble.	Coursed. Uncoursed.	None.	Quarry-faced.
	Dry.	No mortar.	May be any of the above.	May be any of the above.
Concrete.	Broken stone. Gravel. Other material.			
Brick.	Face. No. 1 paving. No. 2 paving. Common.	English bond. Flemish bond.		

* Recommended by American Railway Engineering and Maintenance of Way Association.

Dimension Stone.—A block of stone cut to specified dimensions.

Coping.—A top course of stone or concrete, generally slightly projecting, to shelter the masonry from the weather, or to distribute the pressure from exterior loading.

Arch Masonry.—That portion of the masonry in the arch ring only, or between the intrados and the extrados.

Ashlar Masonry.—Masonry built of ashlar blocks.

Range Masonry.—Masonry in which the various courses are laid up with continuous horizontal beds.

Broken Range Masonry.—Masonry in which the bed joints are parallel but not continuous.

Square-Stone Masonry.—Masonry in which the stones are roughly squared and roughly dressed on beds and sides.

Rubble Masonry.—Masonry composed of squared or roughly squared stones, or rubble of irregular size or shape.

Dry Wall.—A masonry wall in which stones are built up without the use of mortar.

CONCRETE.

Concrete.—A compact mass of broken stone, gravel or other suitable material assembled together with cement mortar and allowed to harden.

Rubble Concrete.—Concrete in which rubblestone are imbedded.

Reinforced Concrete.—Concrete which has been reinforced by means of metal in some form, so as to give the concrete elasticity and increased strength.

BRICK.

Brick.—No. 1.—Hard burned brick, absorption not to exceed 2 per cent by weight.

Brick.—No. 2.—Softer and lighter brick than No. 1, absorption not to exceed 6 per cent by weight.

CEMENT.

Cement.—A preparation of calcined clay and limestone, or their equivalents, possessing the property of hardening into a solid mass when moistened with water. This property is exercised under water, as well as in the air. Cements are divided into four classes: Portland, Natural, Puzzolan and Silica cement. (See each.)

Portland Cement.—This term is applied to the finely pulverized product resulting from the calcination to incipient fusion of an intimate mixture of properly proportioned argillaceous and calcareous materials, and to which no addition greater than 3 per cent has been made subsequent to calcination.

Natural Cement.—This term shall be applied to the finely pulverized product resulting from the calcination of an argillaceous limestone at a temperature only sufficient to drive off the carbonic acid gas.

Puzzolan.—An intimate mixture of pulverized granulated furnace slag and slaked lime without further calcination which possesses the hydraulic qualities of cement.

Silica Cement (sand cement).—A mixture of clean sand and Portland cement ground together.

Descriptive Words.

Arris.—An external angle, edge or ridge.

Ashlar.—A squared or cut block of stone with rectangular dimensions.

Axed.—Dressed so as to cover the surface of a stone with chisel marks which are nearly or quite parallel.

Backing.—That portion of a masonry wall or structure built in the rear of the face. It must be attached to the face and bonded with it. It is usually of a cheaper grade of masonry than the face.

Batter.—The slope or inclination of the face from a vertical line.

Bed.—The top and bottom of a stone. (See Course Bed; Natural Bed; Foundation Bed.)

Bed Joint.—A horizontal joint, or one perpendicular to the line of pressure.

Beton.—(See Concrete.)

Block Rubble.—Large blocks of building stone as they come from the quarry. (See Rubble.)

Bond.—The mechanical disposition of stone, brick or other building blocks by overlapping to break joints.

Build.—A vertical joint.

Bush-Hammered.—A surface produced by removing the roughness of stone with a bush-hammer.

Centering.—A temporary support used in arch construction. (Also called Centers.)

Course.—Each separate layer in stone, concrete or brick masonry.

Course Bed.—Stone, brick or other building material in position, upon which other material is to be laid.

Crandalled.—Dressed with a crandalling tool, producing the same effect as fine-pointed.

Draft.—A line on the surface of a stone cut to the breadth of the chisel.

Draft Stones.—Stones on which the face is surrounded by a draft, the space inside the draft being left rough.

Dressing.—The finish given to the surface of stones or to concrete.

Expansion Joint.—A vertical joint or space to allow for temperature changes.

Extrados.—The upper or convex surface of an arch.

Face.—The exposed surface in elevation.

Facing.—In concrete: (1) A rich mortar placed on the exposed surfaces to make a smooth finish. (2) Shovel facing by working the mortar of concrete to the face.

Final Set.—A stage of the process of setting marked by certain hardness. (See Cement Specifications.*)

Fine-pointed.—Dressed by fine point to smoother finish than by rough point.

Flush.—(Adj.) Having the surface even or level with an adjacent surface. (Verb.) (1) To fill. (2) To bring to a level. (3) To force water to the surface of mortar or concrete by compacting or ramming.

* Standard specifications for cement adopted by the American Society for Testing Materials, American Society of Civil Engineers and the American Railway Engineering and Maintenance of Way Association. Copies of these specifications are furnished gratis by the Universal Portland Cement Co., Chicago, Ill.

Footing.—A projecting bottom course.

Forms.—Temporary structures for holding concrete in desired shape.

Foundation.—(1) That portion of a structure, usually below the surface of the ground, which distributes the pressure upon its support. (2) Also applied to the natural support itself; rock, clay, etc.

Foundation Bed.—The surface on which a structure rests.

Grout.—A thin mortar either poured or applied with a brush.

Header.—A stone which has its greatest length at right angles to the face of the wall, and which bonds the face stones to the backing.

Initial Set.—An early stage of the process of setting, marked by certain hardness. (See Cement Specifications.*)

Intrados.—The inner or concave surface of an arch.

Joint.—The narrow space between adjacent stones, bricks or other building blocks, usually filled with mortar.

Lagging.—Strips used to carry and distribute the weight of an arch to the ribs or centering during its construction.

Leveller.—A small rectangular stone, not less than four to six inches thick, used in broken range work to complete the bed for a stone in the course above and give it proper bond. Sometimes called jumper or dutchman.

Lock.—Any special device or method of construction used to secure a bond in work.

Mortar.—A mixture of sand, cement or lime and water, used to cement the various stones or brick in masonry or to cover the surface of same.

Natural Bed.—The surface of a stone parallel to its stratification.

Paving.—Regularly placed stone or brick forming a floor.

Pean-hammered.—(See Axed.)

Pinner.—A spall or small stone used to wedge up a stone and give it better bearing.

Pitched-faced.—Having the arris clearly defined by a line beyond which the rock is cut away by the pitching chisel so as to make approximately true edges.

Pointing.—Filling joints or defects in the face of a masonry structure.

Quarry-faced.—Stone faced as it comes from the quarry.

Random Range Masonry.—(See Broken Range Masonry.)

Riprap.—Rough stone of various sizes placed compactly or irregularly to prevent scour by water.

Rough-pointed.—Dressed by pick or heavy point until the projections vary from one-half to one inch.

Rubbed.—A fine finish made by rubbing with grit or sandstone.

Rubble.—Field stone or rough stone as it comes from the quarry. When it is of large or massive size it is termed block rubble.

Set (noun).—The change from a plastic to a solid or hard state.

Soffit.—The under side of a projection.

Spall (noun).—A chip or small piece of stone broken from a large block.

Stretcher.—A stone which has its greatest length parallel to the face of the wall.

Tooth-axed.—Dressed by a method of fine pointing.

Voussoirs.—The stones, blocks of concrete or other material forming the arch ring.

SPECIFICATIONS FOR STONE MASONRY.*

General.

1. *Stone.*—Stone masonry shall be built of the kinds specially designated, with such arrangements of courses and bond as shown on the drawings or as directed. The stone shall be hard and durable, free from seams or other imperfections, of approved quality and shape, and in no case have less bed than rise, and shall be laid on their broadest beds, well bonded and solidly bedded. When liable to be affected by freezing, no unseasoned stone shall be laid.

2. *Dressing.*—Dressing must be the best of the kind specified for each class of work.

3. *Beds.*—Beds and joints or builds must be square with each other, and dressed true and out of wind. Hollow beds will not be allowed.

4. *Dressing.*—All stone must be dressed for laying on natural beds.

5. *Drafts.*—Margin drafts must be neat and accurate.

6. *Pitching.*—Pitching must be done to true lines and exact batter.

7. *Mixing.*—The sand and cement should be mixed dry and in small batches in proportions as directed, on a suitable platform, which must be kept clean and free from all foreign matter; then water is to be added, and the whole mixed until the mass of mortar is thoroughly homogeneous and leaves the hoe clean when drawn from it. It must not be retempered after it has begun to set.

8. *Laying.*—All stones must be laid on natural beds. Each stone must be settled into place in full bed of mortar.

9. No stone must be dropped or slid over the wall, but must be placed without jarring the stones already laid.

10. No heavy hammering will be allowed on the wall after a course is laid.

11. If a stone becomes loose after the mortar is set, it must be relaid with fresh mortar.

12. Each stone must be cleansed and dampened before laying.

13. *Laying in Freezing Weather.*—Stones must not be laid in freezing weather unless allowed by the engineer. If allowed, they must be freed from ice, snow or frost by warming, and laid in mortar made of heated sand and water, or, with proper precautions, mixed with brine in proportions of one pound of salt to eighteen gallons of water, when the temperature is thirty-two degrees Fahrenheit. Add one ounce of salt for every degree of temperature below thirty-two degrees Fahrenheit.

14.—Stones must be laid to exact lines and levels so as to give the required bond and thickness of mortar in beds and joints.

15. *Pointing.*—Mortar in beds and joints of exposed faces must be removed to a depth of one inch before it has set. No pointing shall be done until the wall is complete and mortar set, nor when frost is in the stone. Wet the joints and fill again with mortar made of equal parts sand and Portland cement. It must be pounded in with "set-in" or calking tool, and finished with a beading tool the width of the joint, used with a straight edge.

* Adopted by the American Railway Engineering and Maintenance of Way Association in 1906.

Classification.

16. Stone masonry will be classified under the following heads: Bridge and Retaining Wall Masonry, Arch Masonry, Culvert Masonry and Dry Masonry.

Bridge and Retaining Wall Masonry.

17. *Classes.*—Bridge and retaining wall masonry shall consist of two classes: (a) Ashlar (either coursed or broken coursed), and (b) rubble.

(*a*) Ashlar Bridge and Retaining Wall Masonry.

18. *Stone.*—In ashlar bridge and retaining wall masonry (either coursed or broken coursed), the stone must be large and well proportioned.

19. *Courses.*—No course to be less than 14 inches nor more than 30 inches thick, the thickness of courses to diminish regularly from bottom to top.

20. *Dressing.*—The beds and joints or builds of face stones shall be fine pointed, so that the mortar layer shall not exceed one-half an inch in thickness when the stones are laid.

21. *Joints.*—Joints in face stones must be full to the square for a depth equal to at least one-half the height of the course, but in no case less than 12 inches.

22. *Surface Finish.*—The exposed surface of each face stone will be rock faced, and the edges pitched to true lines and exact batter; the face to have no projections over 3 inches beyond the pitch lines.

23. *Chisel Draft.*—A chisel draft one and one-half inches wide shall be cut at each exterior corner.

24. *Handling.*—No holes for stone hooks will be permitted to show in exposed surfaces. They must be handled with clamps, keys, lewis or dowels.

25. *Stretchers.*—Stretchers shall be not less than 4 feet long, and to have at least one and one-fourth times as much bed as thickness of the course.

26. *Headers.*—Headers shall be not less than 4 feet in length. They shall occupy one-fifth of the face of the wall, and no header shall have less than 18 inches width of face, and where the course exceeds 18 inches in height the width of face shall not be less than the height of course. Headers shall hold the size in the heart of the wall that they show on the face, and be so arranged that a header in a superior course shall be placed between two headers in a course below; but no header shall be laid over a joint, and no joint shall occur over a header. They shall be similarly disposed in the back of the wall, interlocking with those in the face when the thickness of the wall will admit. When the wall is too thick to admit of such arrangement, stones of not less than 4 feet in length shall be placed transversely in the heart of the wall to bond the two opposite sides of it.

27. *Backing.*—Backing shall be large, well shaped stone, roughly bedded and jointed, the bed joints not to exceed 1 inch and vertical joints generally not to exceed 2 inches. No part or portion of vertical joints shall have a greater dimension than 6 inches, which void shall be thoroughly filled with spalls full bedded in cement mortar or filled with concrete. At least one-half of the backing stones shall be of the same size and character as the face stone and with parallel beds.

28. When face stone is backed with two courses, neither course shall be less than 8 inches thick.

29. When the wall is 3 feet thick or less, the face stone shall pass entirely through and no backing be allowed.

30. If the engineer so directs, the backing may be entirely of concrete, or back laid with headers and stretchers, as specified above, and heart of wall filled with concrete.

31. The bond of stone on face, back and heart of wall shall not be less than 12 inches. Backing shall be laid to break joints with the face stone and with one another.

32. *Coping.*—Coping will be dimension stone, holding full size throughout, proportioned for its loading, as marked on the drawings.

33. The beds, joints and top will be fine pointed.

34. The location of joints will be determined by the position of the bed plates, and must be shown on the drawings.

35. When required, in the judgment of the engineer, coping stones, stones in the wings of abutments and the stones on piers shall be secured together with iron cramps or dowels, their position being indicated by the engineer.

(*b*) Rubble Bridge and Retaining Wall Masonry.

36. *Stones.*—Rubble bridge and retaining wall masonry will consist of stones roughly squared, laid in irregular courses. The beds must be parallel, roughly dressed and lie horizontally in the wall. The bottom shall be large, selected flat stones. The wall must be compactly laid, having at least one-fifth the surface of the back and face headers, so arranged as to interlock, having all the spaces in the heart of the wall filled with suitable stones and spalls, thoroughly bedded in cement mortar or filled with concrete. The face joints must not be more than 1 inch in thickness.

Arch Masonry.

37. Arch masonry shall consist of the arch ring only, or that portion between the intrados and extrados, and shall be of two classes: (a) Ashlar Arch Masonry, and (b) Rubble Arch Masonry.

(*a*) Ashlar Arch Masonry.

38. *Voussoirs.*—The voussoirs must be full size throughout, and must have bond not less than thickness of the stone and dressed true to templet.

39. The number of courses and depth of voussoirs will be shown on the drawings.

40. *Joints.*—The joints of the voussoirs and the intrados shall be fine pointed. Mortar joints shall not exceed three-eighths of an inch.

41. *Finish.*—The exposed surface of the ring stone shall be smooth or rock faced, with a marginal draft.

42. Voussoirs shall be carried up simultaneously from both bench walls.

43. *Backing.*—Backing may consist of large stones shaped to fit the arch, bonded to the spandrel and laid in full beds of mortar. Concrete may also be used for backing.

44. *Waterproofing.*—If waterproofing is required, a thin coat of mortar or

grout shall be applied for a finishing coat, upon which shall be placed a covering of suitable waterproofing material.

45. *Centers.*—Centers shall not be struck until directed.

46. *Miscellaneous Walls.*—Bench walls, piers, spandrels, parapets wing walls and copings will be built under the specifications for ashlar bridge and retaining wall masonry.

(*b*) Rubble Arch Masonry.

47. *Voussoirs.*—The voussoirs must be full size throughout, and must have bond not less than thickness of the stone.

48. The depth of voussoirs will be shown on the drawings.

49. The beds need only be roughly dressed so as to bring them to radial planes.

50. *Joints.*—Mortar joints shall not exceed 1 inch.

51. *Finish.*—The exposed surface of the ring stones shall be rock faced and the edges pitched to true lines.

52. Voussoirs shall be carried up simultaneously from both bench walls.

53. *Backing.*—Backing may consist of large stones shaped to fit the arch, bonded to the spandrel and laid in full beds of mortar. Concrete may also be used for backing.

54. *Waterproofing.*—If waterproofing is required, a thin coat of mortar or grout shall be applied for a finishing coat, upon which shall be placed a covering of suitable waterproofing material.

55. *Centers.*—Centers shall not be struck until directed.

56. *Miscellaneous Walls.* Bench walls, piers, spandrels, parapets and wing walls will be built under the specifications for rubble bridge and retaining wall masonry.

Culvert Masonry.

57. *Stone.*—Culvert masonry must be laid in cement mortar. The character of stone used and quality of work must be similar to that specified for rubble bridge and retaining wall masonry.

58. *Headers.*—One-half the top stones of the side walls must extend entirely across the wall.

59. *Cover Stones.*—The covering must be of sound, strong stone, at least 12 inches thick, or as shown on drawings. They must be roughly dressed so as to make close joints with each other, and must lap their whole width at least 12 inches over the side walls. They must be doubled under high embankments, as directed by the engineer or shown on the drawings.

60. *End Walls.*—The end walls must be covered with suitable coping.

61. *Dry Masonry.*—Dry masonry will include dry retaining walls and slope walls.

Dry Retaining Walls.

62. Dry retaining walls will include all dry rubble work for retaining embankments or similar work.

63. *Stone.*—Flat stone at least twice as wide as thick will be used. Beds and joints to be roughly dressed square to each other and to face of stone.

64. *Joints.*—Joints not to exceed three-fourths of an inch.

65. *Sizes of Stone.*—The different sizes of stone must be evenly distributed

over the whole face of wall, generally keeping the largest stone in the lower part of the wall.

66. *Laying.*—The work shall be well bonded and present a reasonably true and smooth surface, free from holes or projections. This wall is double faced and self-sustaining.

SLOPE WALLS.

67. *Stone.*—Slope walls shall be built of such thickness and slope as may be required by the engineer. No stone shall be used in their construction which does not reach through the wall. Stones to be placed at right angles to the slopes. This wall is single faced and built with steep slopes simultaneously with the embankment which it is to protect.

STANDARD SPECIFICATIONS FOR PORTLAND CEMENT CONCRETE.*

1. *Cement.*—Cement shall be Portland, either American or foreign, which will meet the requirements of the standard specifications.

2. *Sand.*—Sand shall be clean, sharp, coarse, and of grains varying in size. It shall be free from sticks and other foreign matter, but it may contain clay or loam not to exceed five (5) per cent. Crusher dust, screened to reject all particles over one-quarter (¼) in. in diameter, may be used instead of sand if approved by the engineer.

3. *Stone.*—Stone shall be sound, hard and durable, crushed to sizes not exceeding two inches in any direction. For reinforced concrete sizes usually are not to exceed three-quarters (¾) in. in any direction, but may be varied to suit character of reinforcing material.

4. *Gravel.*—Gravel shall be composed of clean pebbles of hard and durable stone of sizes not exceeding two inches in diameter, and shall be free from clay and other impurities except sand. When containing sand in any considerable quantity, the amount of sand per unit of volume of gravel shall be determined accurately, to admit of the proper proportion of sand being maintained in the concrete mixture.

5. *Water.*—Water shall be clean and reasonably clear, free from sulphuric acid or strong alkalies.

6. *Mixing by Hand.*—(*a*) Tight platforms shall be provided of sufficient size to accommodate men and materials for the progressive and rapid mixing of at least two batches of concrete at the same time. Batches shall not exceed one cubic yard each, and smaller batches are preferable, based upon a multiple of the number of sacks of cement to the barrel.

(*b*) Spread the sand evenly upon the platform, then the cement upon the sand and mix thoroughly until of an even color. Add all the water necessary to make a thin mortar and spread again; add the gravel if used, and finally the broken stone, both of which, if dry, should first be thoroughly wet down. Turn the mass with shovels or hoes until thoroughly incorporated, and all the gravel

* Adopted by the American Railway Engineering and Maintenance of Way Association, 1904.

and stone is covered with mortar; this will probably require the mass to be turned four times.

(*c*) Another approved method, which may be permitted at the option of the engineer in charge, is to spread the sand, then the cement and mix dry, then the gravel or broken stone; add water and mix thoroughly as above.

7. *Mixing by Machine.*—A machine mixer shall be used wherever the volume of work will justify the expense of installing the plant. The necessary requirements for the machine will be that a precise and regular proportioning of materials can be controlled and that the product delivered shall be of the required consistency and thoroughly mixed.

8. *Consistency.*—The concrete shall be of such consistency that when dumped in place it will not require much tamping. It shall be spaded down and tamped sufficiently to level off, and the water should rise freely to the surface.

9. *Forms.*—(*a*) Forms shall be well built, substantial and unyielding, properly braced or tied together by means of wire or rods, and shall conform to lines given.

(*b*) For all important work, the lumber used for face work shall be dressed on one side and both edges to a uniform thickness and width, and shall be sound and free from loose knots, secured to the studding or uprights in horizontal lines.

(*c*) For backings and other rough work undressed lumber may be used.

(*d*) Where corners of the masonry and other projections liable to injury occur, suitable moldings shall be placed in the angles of the forms to round or bevel them off.

(*e*) Lumber once used in forms shall be cleaned before being used again.

(*f*) The forms must not be removed within thirty-six hours after all the concrete in that section has been placed. In freezing weather they must remain until the concrete has had a sufficient time to become thoroughly set.

(*g*) In dry but not freezing weather, the forms shall be drenched with water before the concrete is placed against them.

10. *Disposition.*—(*a*) Each layer should be left somewhat rough to insure bonding with the next layer above; and, if it be already set, shall be thoroughly cleaned and scrubbed with coarse brushes and water before the next layer is placed upon it.

(*b*) Concrete shall be deposited in the molds in layers of such thickness and position as shall be specified by the engineer in charge. Temporary planking shall be placed at ends of partial layers, so that none shall run out to a thin edge. In general, excepting in arch work, all concrete must be deposited in horizontal layers throughout.

(*c*) The work shall be carried up in sections of convenient length, and each section completed without intermission.

(*d*) In no case shall work on a section stop within 18 inches of the top.

(*e*) Concrete shall be placed immediately after mixing, and any having an initial set shall be rejected.

11. *Expansion Joints.*—(*a*) In exposed work, expansion joints may be provided at intervals of 30 to 100 ft., as the character of the structure may require.

(*b*) A temporary vertical form or partition of plank shall be set up and the section behind completed as though it were the end of the structure. The partition shall be removed when the next section is begun, and the new concrete

placed against the old without mortar flushing. Locks shall be provided if directed or called for by the plans.

(*c*) In reinforced concrete the length of these sections may be materially increased at the option of the engineer.

12. *Facing.*—(*a*) The facing will be made by carefully working the coarse material back from the form by means of a shovel, bar or similar tool, so as to bring the excess mortar of the concrete to the face.

(*b*) About one inch of mortar (not grout) of the same proportions as used in the concrete may be placed next to the forms immediately in advance of the concrete.

(*c*) Care must be taken to remove from the inside of the forms any dry mortar in order to secure a perfect face.

13. *Proportioning.*—The proportion of the materials in the concrete shall be as specifically called for by the contract, or as set forth herein, upon the lines left for that purpose; the volume of cement to be based upon the actual cubic contents of one barrel of specified weight.

Structure.	Parts by Volume.			
	Cement.	Sand.	Gravel.	Broken Stone.

14. *Finishing.*—(*a*) After the forms are removed, which should generally be as soon as possible after the concrete is sufficiently set, any small cavities or openings in the face shall be neatly filled with mortar, if necessary in the opinion of the engineer. Any ridges due to cracks or points in the lumber may be rubbed down with a chisel or wooden float. The entire face may then be washed with a thin grout of the consistency of whitewash, mixed in the same proportion as the mortar of the concrete. The wash should be applied with a brush. The earlier the above operations are performed the better will be the result.

(*b*) The tops of bridge seats, pedestals, copings, wing walls, etc., when not finished with natural stone coping, shall be finished with a smooth surface composed of one part cement to two parts of granite, or other suitable screenings or sand, applied in a layer one-half (½) to one (1) in. thick. This must be put in place with the last course of concrete.

15. *Waterproofing.*—Where waterproofing is required, a thin coat of mortar or grout shall be applied for a finishing coat, upon which shall be placed a covering of suitable waterproofing material.

16. *Freezing Weather.*—Ordinarily concrete to be left above the surface of the ground will not be constructed in freezing weather. Portland cement con-

crete, however, may be built under these conditions by special instructions. In this case the sand, water and broken stone shall be heated, and in severe cold salt shall be added in the proportion of about two pounds per cubic yard.

17. *Reinforced Concrete.*—Where concrete is deposited in connection with metal reinforcing, the greatest care must be taken to insure the coating of the metal with cement, and the thorough compacting of the concrete around the metal. Wherever it is practicable the metal should be placed in position first. This can usually be done in the case where the metal occurs in the bottoms of the forms by supporting the same on transverse wires, or otherwise, when the bottoms of the forms can be flushed with cement mortar, so as to get the mortar under the metal at the same time, and the concrete deposited immediately afterward. The mortar for flushing the bars should be composed of one part cement and two parts sand. The metal used in the concrete shall be free from dirt, oil or grease. All mill scale should be removed by hammering the metal, or preferably by pickling the same in a weak solution of muriatic acid. No salt will be used in reinforced concrete when laid in freezing weather.

STEEL PIERS.—Steel piers are made (1) of steel sections used as piles, (2) in the form of steel bents supported on masonry pedestals, or (3) of tubular or oblong steel shells filled with concrete.

Steel Piles.—A steel pile bent made of two angles "starred," is shown in Fig. 224. This makes a fairly satisfactory pier for short

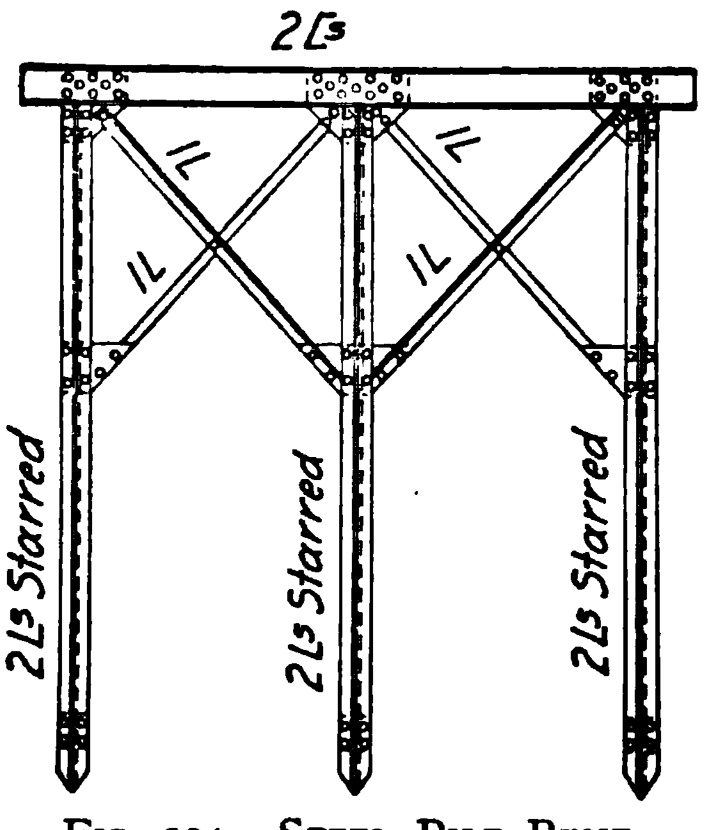


FIG. 224. STEEL PILE BENT.

beam spans. This pier should never be used to carry the earth filling. Steel I beams are frequently used in the place of angles.

Steel Bents.—Steel bents are made of two columns firmly braced together in a plane at right angles to the axis of the bridge. The most

common column for the steel bents is composed of two channels laced on both sides, although Z-bar columns are very satisfactory for heavy loads.

Steel Tubular Piers.—Steel tubular piers are made of steel plates riveted together and filled with concrete. Where the piers are founded on soft material, piles are driven in the bottom of the tube, the piles being sawed off below the water line. The piles should extend at least two diameters of the tube above the bottom. The tubes are braced transversely by means of struts and tension diagonals above high water and by diaphragm bracing below high water. Where the piers will be subject to blows from floating drift or logs they should be protected by a timber cribwork or other device.

Cooper's Standards.—The tubular piers in Fig. 225 are from Cooper's "General Specifications for Foundations and Substructures

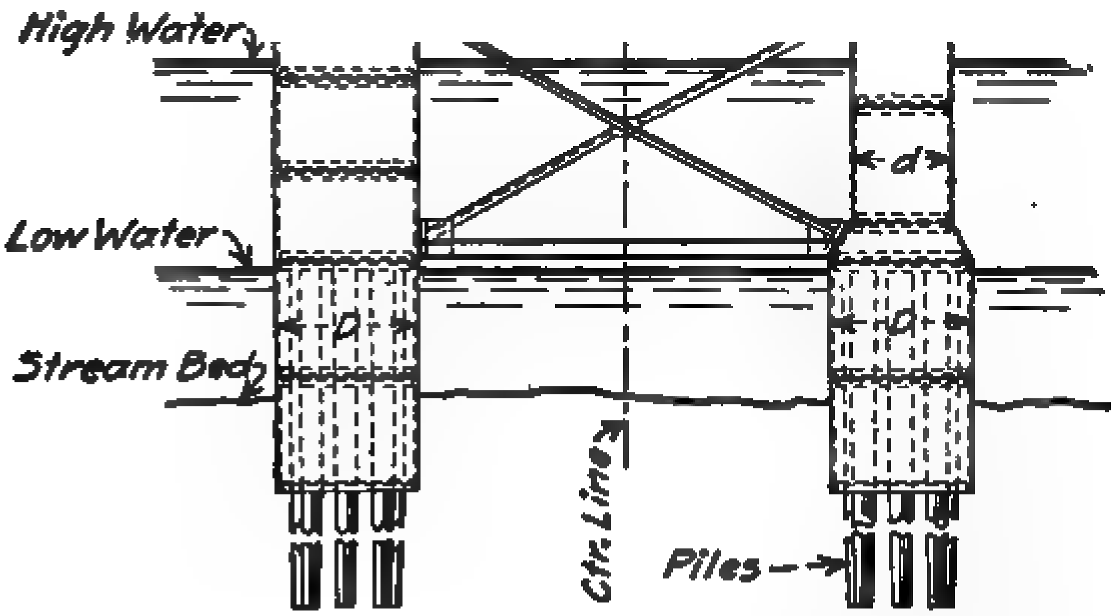


FIG. 225. STEEL TUBULAR PIERS. COOPER'S STANDARD'S.

for Highway and Electric Railway Bridges." Cooper specifies a minimum thickness of $\frac{3}{8}''$ for plates below and $\frac{1}{4}''$ above the high water.

The minimum sizes of tubular piers are as given in Table LII.

A steel tubular pier with a timber crib protection is given in Fig. 226. The crib is filled with loose rock.

An oblong steel shell pier, as designed by Cooper, is given in Fig.

TABLE LII.

MINIMUM SIZES OF STEEL TUBULAR PIERS, COOPER'S STANDARDS, FIG. 225.

Span in Feet.	Highway and Single Track Electric Railway.			Double Track Electric Railway.		
	Minimum. Top d.	Diameter. Bot. D.	No. of Piles.	Minimum. Top d.	Diameter. Bot. D.	No. of Piles.
50	2' 10"	3' 4"	4	3' 4"	4' 4"	8
75	3 4	3 9	5	3 10	5 6	10
100	3 8	4 2	6	4 6	6 0	10
125	4 0	4 7	8	4 10	6 4	12
150	4 4	5 0	9	5 2	7 0	12
175	4 8	5 6	10	5 6	7 6	15
200	5 0	5 10	11	5 10	8 0	15
250	5 6	6 4	12	6 4	9 0	19

227. The center of the truss is to come $a/2 + 1$ ft. from the end of the pier. The width a, as specified by Cooper, is given in Table LIII.

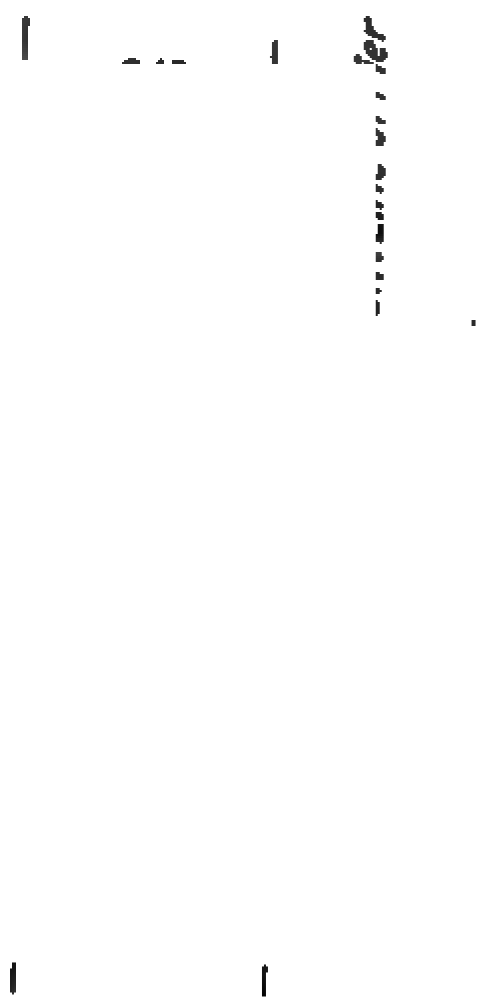


FIG. 226. STEEL TUBULAR PIER WITH TIMBER CRIB PROTECTION. COOPER'S STANDARDS.

American Bridge Company Standards.—The American Bridge Company's standard tubular piers are shown in Fig. 228. The mini-

mum diameters for a height of 15 feet to carry a single span are specified as in Table LIV.

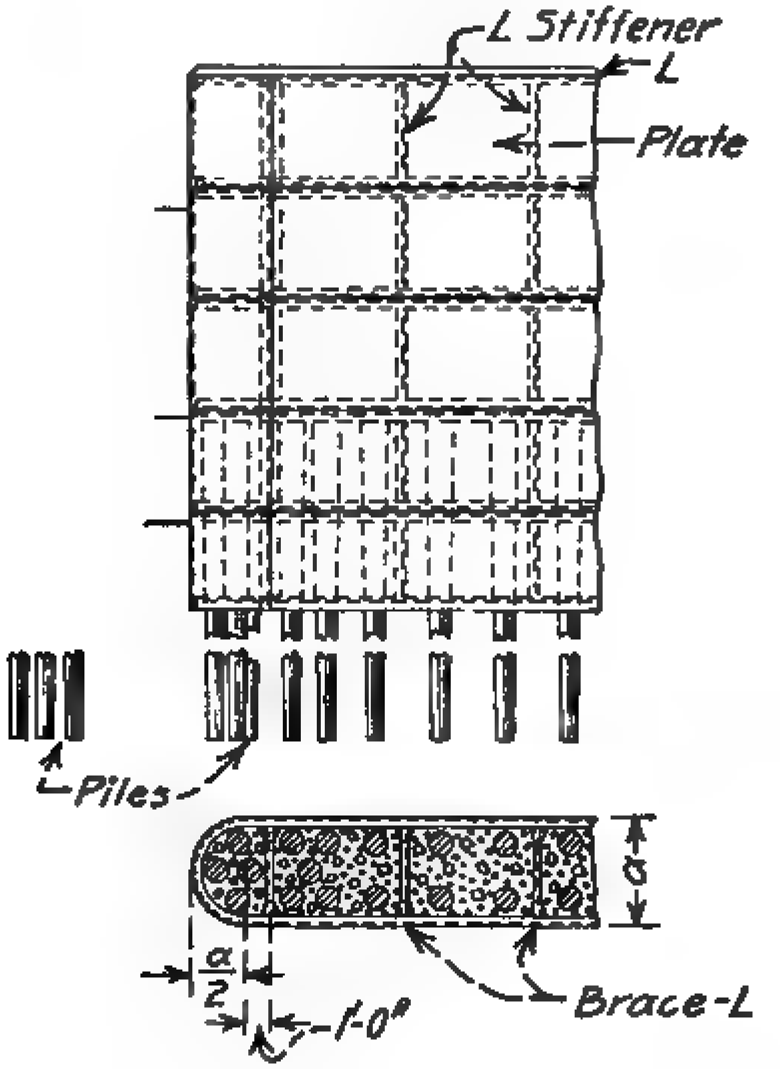


FIG. 227. OBLONG STEEL TUBULAR PIER. COOPER'S STANDARDS.

TABLE LIII.

MINIMUM SIZES OF OBLONG STEEL PIERS, COOPER'S STANDARDS.

Span in Feet.	Width a, Fig. 227.	
	Highway and Single Track Electric Railway.	Double Track Electric Railway.
50	2' 10"	3' 4"
75	3 4	4 0
100	3 8	4 6
125	4 0	4 10
150	4 4	5 2
175	4 8	5 6
200	5 0	5 10
250	5 6	6 4

The tubes are made of 62½-inch plates plus one variable plate. The weight of the steel in the tubes per lineal foot for different thicknesses

is given in Fig. 229. In calculating the weight of a pier add one foot to the length of each tube before entering the diagram. The weight of the concrete in two tubes is given in Fig. 229. The concrete is assumed

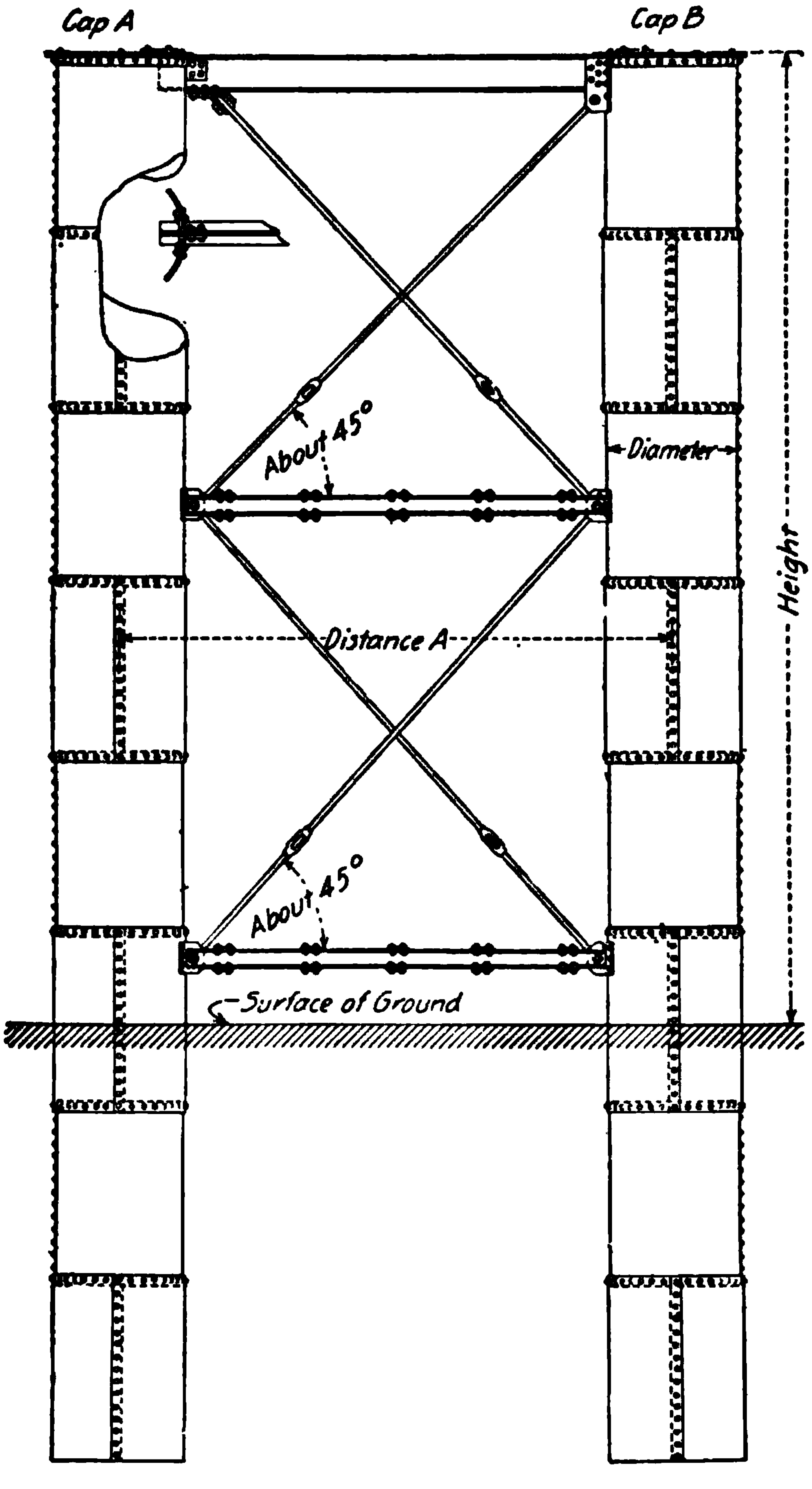


FIG. 228. STEEL TUBULAR PIERS. (AMERICAN BRIDGE CO.)

to fill the tube, the space occupied by piles should be deducted.

The number of piles required for different diameters of tubes is given in Table LV.

The number of piles required for large tubes agrees quite closely

with Cooper's specifications, but the number for small tubes is very much less.

Pier Beams.—The sizes of pier beams required for different panel lengths and clear distance between tubes in feet are given in Fig. 230.

TABLE LIV.

MINIMUM DIAMETERS OF STEEL TUBULAR PIERS FOR A HEIGHT OF 15 FEET TO CARRY A SINGLE SPAN. AMERICAN BRIDGE COMPANY STANDARDS.

SPAN IN FEET.	DIAMETER IN INCHES.
25	18
50	21
75	24
100	27
125	30
150	33
175	36
200	42

Increase the diameter 3 ins. for each additional 5 ft. in height.

TABLE LV.

NUMBER OF PILES REQUIRED FOR TUBULAR PIERS. AMERICAN BRIDGE COMPANY STANDARDS.

DIAMETER OF TUBE IN INCHES.	NUMBER OF PILES IN ONE TUBE.	DIAMETER OF TUBE IN INCHES.	NUMBER OF PILES IN ONE TUBE.
18	1	48	4
24	1	54	5
30	1	60	6
33	1	66	7
36	2	72	8
42	3	78	10
45	3	84	13

The pier beam should be assumed as one foot longer than the clear distance between the tubes, in calculating the weight of the beams.

Pier Bracing.—The pier bracing for piers supporting the ends of two spans are given in Fig. 231. If the spans are unequal in length, enter the diagram with one-half of the algebraic sum of the spans. For example, for a pier carrying a 75-ft. and a 125-ft. span, enter the diagram with a span of 100 ft. *Steel tubular piers should never be used for end abutments carrying a fill.*

In calculating the weight of the diagonal bars the length of the bar should be multiplied by the weight per foot as obtained from a hand-

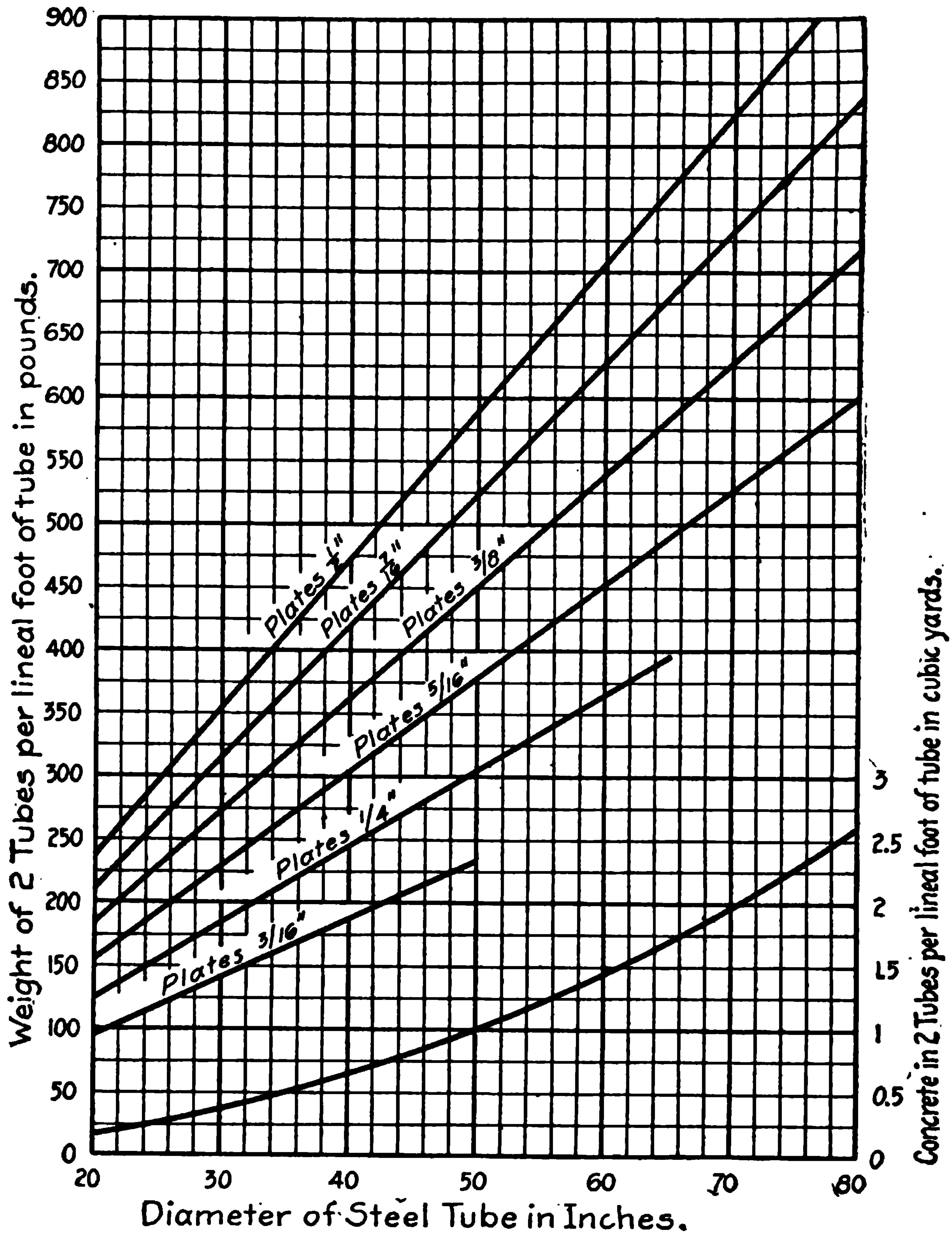


FIG. 229. WEIGHT OF 2 STEEL TUBES PER LINEAL FOOT. (AMERICAN BRIDGE CO.) STANDARDS.

book, and the details for one bar added to the product. In calculating the weight of the struts add one foot to the clear length.

Pier Caps.—Tubular piers may be capped with steel plate caps, may be finished with concrete, or may have a stone pedestal block as in Fig. 232. The weights given above do not include the weights of steel caps.

Details.—The shop details of one tube of a steel pier for a 220-ft.

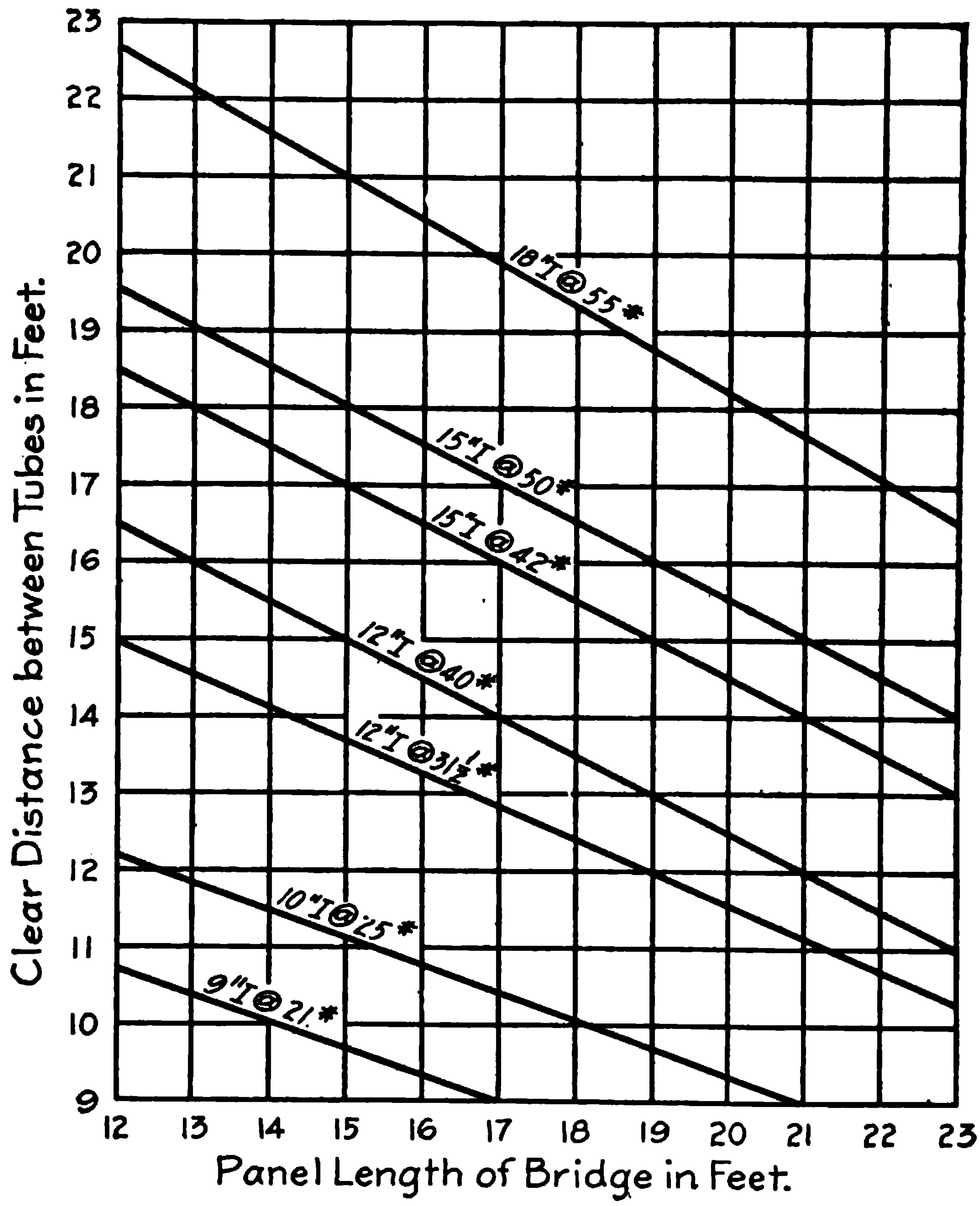


FIG. 230. PIER BEAMS FOR TUBULAR PIERS. (AMERICAN BRIDGE CO.)

highway bridge are shown in Fig. 232. The horizontal joints are lap joints, while the vertical joints are single strap butt joints. This tube was 4′ 3″ in diameter. These tubes were to have 5 piles driven in the bottom, and were then filled with 1–3–6 Portland cement concrete.

The shop details of steel tubular piers for a 95-ft. riveted highway bridge are given in Fig. 166.

A steel tubular pier for a 120-ft. span electric railway bridge is shown in Fig. 233. The lower section of the tube is 8 feet in diameter, while the upper section of the tube is 4 feet in diameter. The details of the bridge carried by these piers are given in Figs. 160 and 161.

Specifications for Steel Tubular Piers.—The plates for the tubes shall be not less than $\frac{1}{4}$ in. thick for tubes up to 30 ins. in diameter, not less than $\frac{5}{16}$ in. for tubes from 30 to 48 ins. in diameter, and $\frac{3}{8}$ in. for tubes from 48 to 72 ins. in diameter. Where the plates are in contact with the soil the thickness shall be increased at least $\frac{1}{16}$ in. For $\frac{5}{16}$ in. plate and less use $\frac{5}{8}$ in rivets; for $\frac{3}{8}$ in. plate and over use $\frac{3}{4}$ in. rivets.

BRACING FOR TUBULAR PIERS.

Bridge Span in Ft.	Diagonal Bracing		Pier Struts.			
	Diam. of Bar	Details of 1 Bar in Lbs.	Roadway 12' to 14'		Roadway 16' to 20'	
			Strut	Wt. per ft. in lbs.	Strut	Wt. per ft. in lbs.
25	$\frac{7}{8}$" □	20	2-4" ⊏ @ $5\frac{1}{4}$#	17	2-4" ⊏ @ $5\frac{1}{4}$#	17
50	$1\frac{1}{8}$" □	30	2-4" ⊏ @ $5\frac{1}{4}$#	17	2-5" ⊏ @ $6\frac{1}{2}$#	19
75	$1\frac{3}{8}$" □	45	2-4" ⊏ @ $5\frac{1}{4}$#	17	2-6" ⊏ @ 8#	22
100	$1\frac{5}{8}$" □	65	2-5" ⊏ @ $6\frac{1}{2}$#	19	2-7" ⊏ @ $9\frac{3}{4}$#	26
125	$1\frac{7}{8}$" □	90	2-6" ⊏ @ 8#	22	2-7" ⊏ @ $9\frac{3}{4}$#	26

FIG. 231. BRACING FOR TUBULAR PIERS. (AMERICAN BRIDGE CO.)

The horizontal seams shall be single lap joints riveted with a pitch of 4 diameters of rivet; while the vertical seams shall preferably be butt riveted with single riveting spaced 4 diameters of rivet, up to 48 inches diameter of tubes, and double riveting with 3 inch spacing for tubes of larger diameter.

The bracing of piers shall be designed to take all the wind forces specified to come on the birdge. Diaphragm webs are to be used up to well above high water for piers located in the stream or where floating materials may find lodgment. Oblong piers shall be braced against inside and outside pressure. Piers exposed to injury from floating logs and drift shall be protected.

The tubes should be painted inside and out with two coats of red lead and linseed oil, or other prescribed paint.

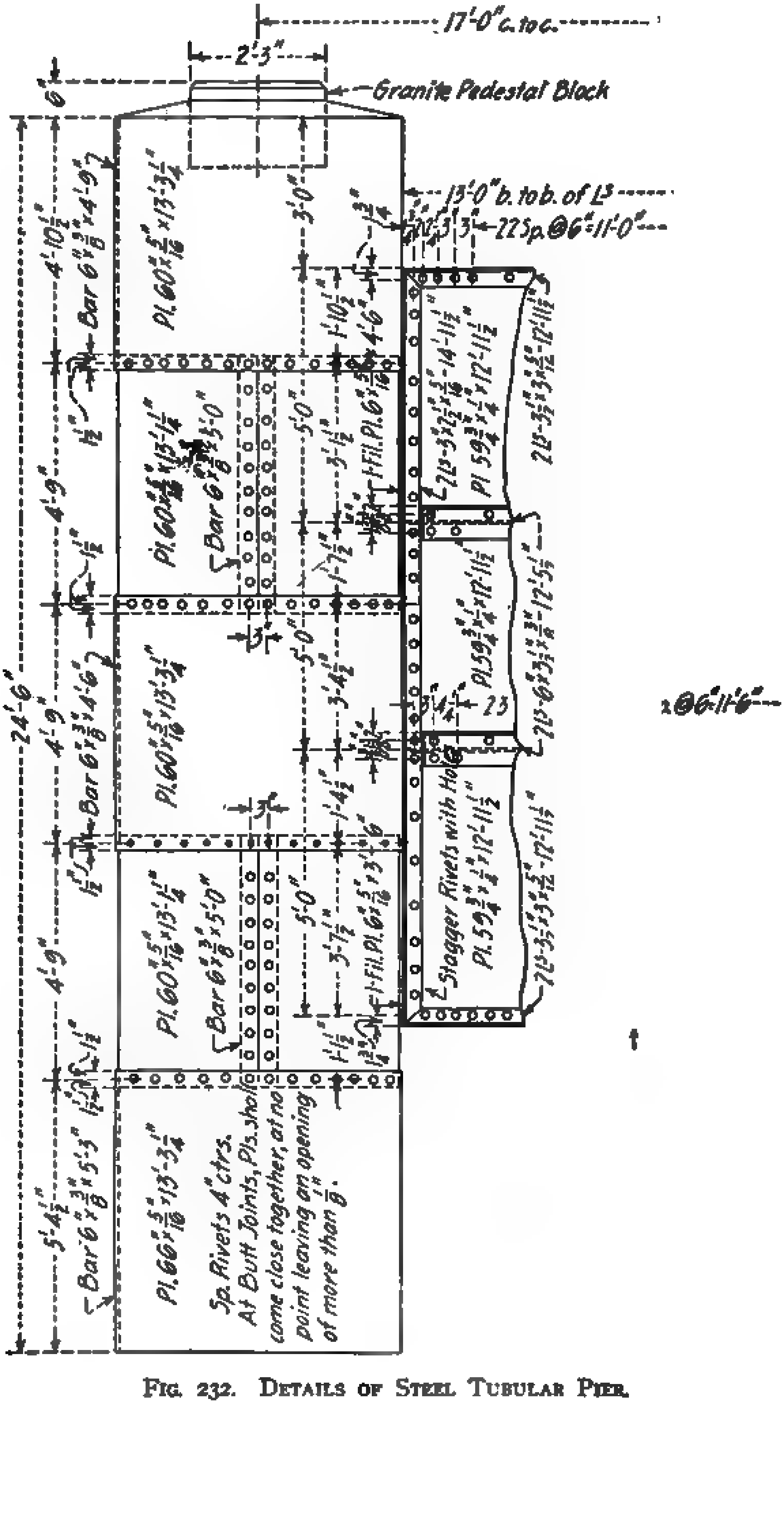


FIG. 232. DETAILS OF STEEL TUBULAR PIER.

The materials and workmanship shall comply with the specifications in Appendix I.

Erection.—Where the bottom will permit, the tubes shall be sunk well below possible scour by loading the tube and excavating the

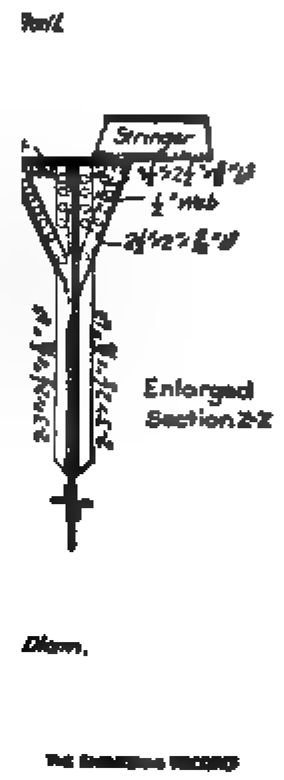


FIG. 233. STEEL TUBULAR PIER FOR 130-FT. SPAN ELECTRIC RAILWAY BRIDGE.

material from the inside. For this purpose a clamshell bucket is very effective. Driving the tube with a pile driver will cut off the rivets in the horizontal seams and will not be permitted. After the tube is sunk, piles are to be driven inside of the steel shell, as closely together as possible, using care to get no pile nearer than 4 to 6 ins to the steel shell. The piles shall be driven to a good refusal, and the tops sawed off below the low water mark and reaching at least 2 diameters of the tube above the bottom. The space inside the tubes shall then be

filled with concrete well tamped. Concrete should not be deposited in running water if possible to prevent it.

Where piers are founded on rock, the tubes are to be anchored to the rock and then filled with concrete. Or cribs may be sunk on the rock and the tube set in a pocket in the crib and resting on the rock. The space outside the tube is then filled with concrete and the tube is filled with concrete in the usual manner.

CHAPTER XVI.

STRESSES IN SOLID MASONRY ARCHES.

Introduction.—An arch is a beam or framework in which the reactions are not vertical for vertical loads. Arches are divided, according to the number of hinges, into three-hinged arches, two-hinged arches, one-hinged arches and arches without hinges or continuous arches. Solid two-hinged and continuous arches constructed of masonry or concrete, only, will be considered in this chapter. For the analysis of a two-hinged arch with spandrel bracing, see the author's "The Design of Steel Mill Buildings," Chapter XIV.

The vertical reactions in arches are the same as in a simple beam having the same loads and span. The horizontal reactions, however, depend upon the deformation of the arch, and are not statically determinate. Two-hinged masonry or concrete arches are rarely used, but the theory of this type will be deduced as preliminary to the theory of the arch with fixed abutments.

STRESSES IN A TWO-HINGED ARCH. Reactions.—The vertical reactions of the two-hinged arch in (a) Fig. 235, are the same

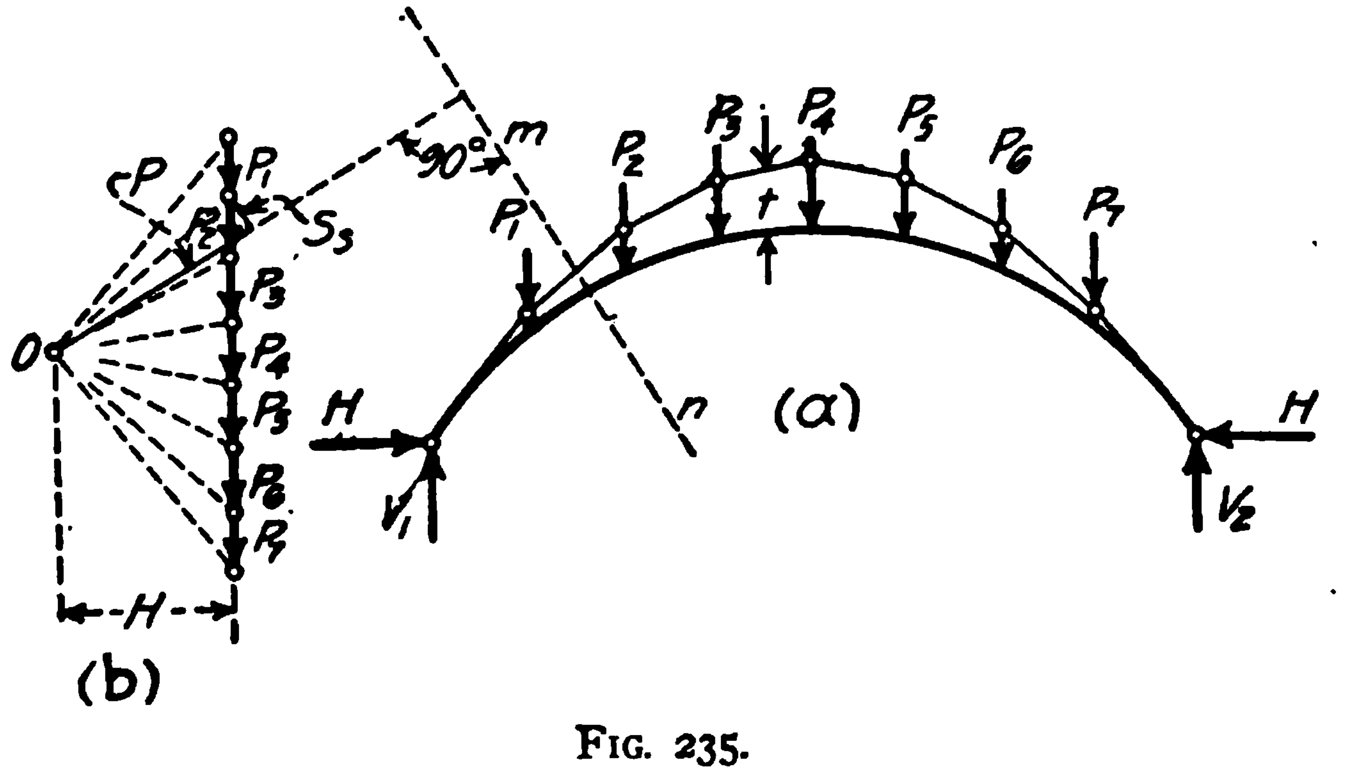


FIG. 235.

as for a simple beam having the same loads and span. The horizontal reactions will be $H = H$, and will be equal to the pole distance of the force polygon in (b) that is used to draw the true equilibrium polygon. The value of H depends upon the elasticity of the arch and is not statically determinate.

Having calculated the vertical reactions V_1 and V_2 by means of moments in the usual manner, and the horizontal components H, as will be described presently, the equilibrium polygon in (a) may be drawn using the force polygon in (b).Fig. 235. The requirements being that the equilibrium polygon must pass through the hinges, and that the force polygon must have a pole distance equal H.

The bending moment at any point in the arch will then be $H \cdot t$, where H is the pole distance of the equilibrium polygon, and t is the intercept from the point at which the moment is to be determined to the string P_3P_4. The shear on the section of the arch m–n will be S_s, while the direct axial stress will be P, as shown in (b) Fig. 235.

Calculation of Horizontal Reaction, H.—Now for equilibrium in the two-hinged arch in Fig. 236, the following conditions must be satisfied: (1) the span must remain constant; and (2) the abutments must maintain the same relative positions. These relations will be expressed in the form of equations of condition.

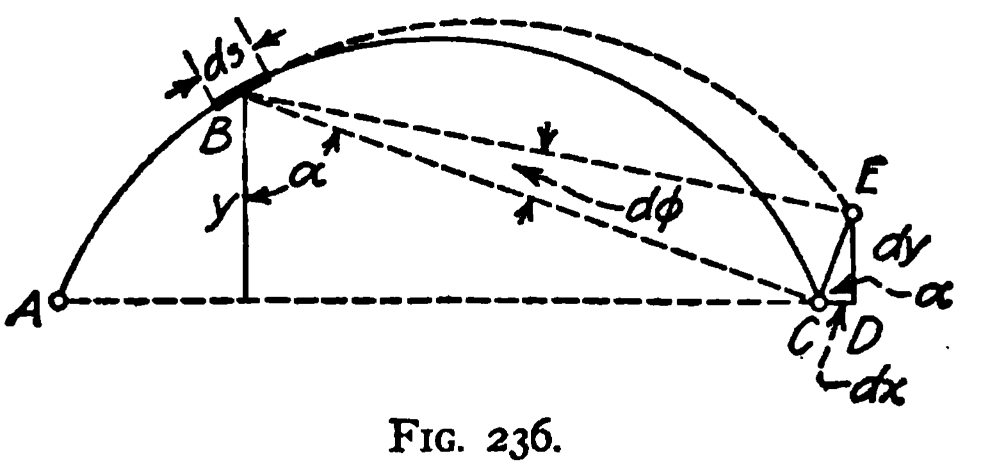


FIG. 236.

In Fig. 236 assume that the arch ring is rigid except the length ds which bends under the action of some external loading. Now the point C will move to E if the arch be not constrained, the horizontal deformation being $CD = dx$, and the vertical deformation being $ED = dy$. The angle $CBE = d\phi$.

Then

$$CE = BC \cdot d\phi$$

$$BC = y \cdot \sec \alpha$$

$$dx = CE \cdot \cos \alpha$$

$$= y \cdot d\phi \qquad (101)$$

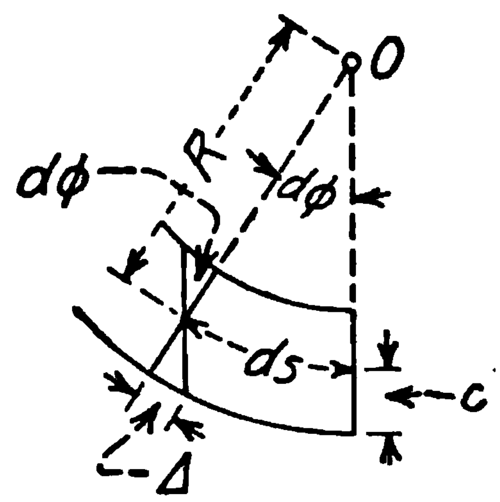


FIG. 237.

Now in a beam, as in Fig. 237, the stresses at any point in the beam will vary as the distance from the neutral axis, and from similar triangles we have

$$R : ds :: c : \Delta,$$

and

$$R \cdot \Delta = c \cdot ds \qquad (102)$$

Now if S is the fiber stress on the extreme fiber, and E is the modulus of elasticity, we have $\Delta : S :: ds : E$, and

$$\Delta \cdot E = S \cdot ds \qquad (103)$$

and solving (102) and (103) for R, we have

$$R \cdot S = E \cdot c \qquad (104)$$

But from the common theory of flexure we have $M \cdot c = S \cdot I$, and substituting in (104)

$$R = E \cdot I / M \qquad (105)$$

Also

$$R \cdot d\phi = ds, \text{ and } d\phi = ds/R \qquad (106)$$

Substituting the value of R as given in (105) in (106) we have

$$d\phi/ds = M/E\cdot I \tag{107}$$

And substituting the value of $d\phi$ as given in (107) in (101)

$$dx = M\cdot y\cdot ds/E\cdot I \tag{108}$$

Now if bending takes place over the entire length of the span of the arch the total horizontal deformation will be

$$\Delta x = \sum \frac{M\cdot y\cdot ds}{E\cdot I} \tag{109}$$

In the two-hinged arch the span is constant and

$$\Delta x = \sum \frac{M\cdot y\cdot ds}{E\cdot I} = 0 \tag{110}$$

Now in Fig. 236

$$CE = BC\cdot d\phi$$
$$BC = x\cdot \csc\alpha$$
$$dy = CE\cdot \sin\alpha$$
$$= x\cdot d\phi$$

and in the same manner as for Δx it can be proved that

$$\Delta y = \sum \frac{M\cdot x\cdot ds}{E\cdot I} = 0 \tag{111}$$

Now either equation (110) or equation (111) will be sufficient to determine the pole distance in Fig. 235.

Now in equation (110) the value of M at any point in the arch will be $M = M' - H\cdot y$, where M' is the bending moment as calculated in a simple beam, H is the horizontal component of the reaction and y is the ordinate of the point in the arch as in Fig. 236.

Inserting the value of M in equation (110), it becomes

$$\sum \frac{(M' - H\cdot y)}{E\cdot I} y\cdot ds = 0,$$

and

$$\sum \frac{M' \cdot y \cdot ds}{E \cdot I} - H \sum \frac{y^2 \cdot ds}{E \cdot I} = 0,$$

and

$$H = \frac{\sum \frac{M' \cdot y \cdot ds}{E \cdot I}}{\sum \frac{y^2 \cdot ds}{E \cdot I}} \qquad (112)$$

In like manner by inserting the value of M in (111), it becomes

$$H = \frac{\sum \frac{M' \cdot x \cdot ds}{E \cdot I}}{\sum \frac{x \cdot y \cdot ds}{E \cdot I}} \qquad (113)$$

Graphic Solution.—Now in Fig. 238 let polygon AEB be a random polygon drawn with an assumed pole distance H'; polygon ADB is the true equilibrium polygon drawn with the true pole distance H; and ACB is the linear arch.

Then the bending moment at C in Fig. 238 will be

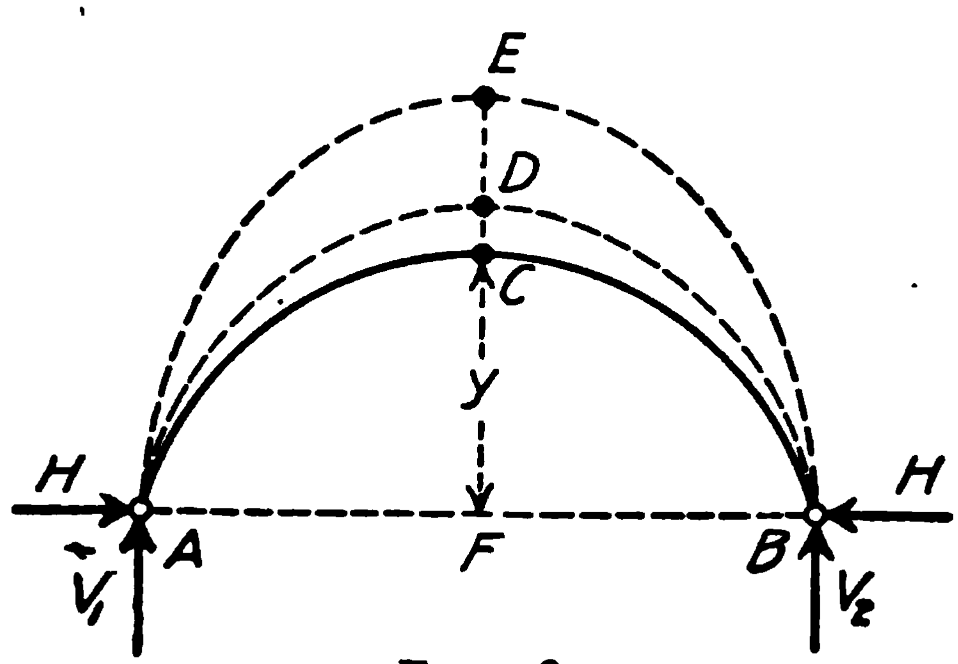


FIG. 238.

$$\begin{aligned} M &= M' - H \cdot y \\ &= H \cdot CD \\ &= H \cdot DF - H \cdot CF \\ &= H \cdot DF - H \cdot y \end{aligned}$$

But DF is not yet known. However, we have the relation that the

ordinates to the two equilibrium polygons are inversely proportional to the pole distances; and

$$EF:DF::H:H',$$

and

$$DF = EF\cdot H'/H = EF\cdot r,$$

where r equals the ratio of the assumed pole distance to the true pole distance.

Then

$$M = EF\cdot H\cdot r - H\cdot y \tag{114}$$

Now substituting the value of M given in (114) in (110)

$$\Sigma\frac{EF\cdot H\cdot r\cdot y\cdot ds}{E\cdot I} - \Sigma\frac{H\cdot y^2\cdot ds}{E\cdot I} = 0,$$

and

$$r = \frac{\Sigma\frac{y^2\cdot ds}{E\cdot I}}{\Sigma\frac{EF\cdot y\cdot ds}{E\cdot I}} \tag{115}$$

In like manner equation (111) becomes

$$r = \frac{\Sigma\frac{x\cdot y\cdot ds}{E\cdot I}}{\Sigma\frac{EF\cdot x\cdot ds}{E\cdot I}} \tag{116}$$

Now in equations (115) and (116) if $E\cdot I$ is a constant, the arch may be divided into segments of equal length; or if $E\cdot I$ is not a constant the arch may be divided into segments for which $ds/E\cdot I$ is a constant, and we may write

$$r = \Sigma y^2/\Sigma EF\cdot y \tag{115'}$$

and

$$r = \Sigma x\cdot y/\Sigma EF\cdot x \tag{116'}$$

Graphic Interpretation of Equations.—Referring to Fig. 238, it will be seen that the numerator in (115′) is the summation of the

products of the ordinates to the arch taken at the centers of the segments into which the arch ring is divided; while the denominator is the summation of the products of the ordinates to the random equilibrium polygon taken at the centers of the segments into which the arch ring is divided, and the distances of the segment from the line AB.

In (116′) the numerator is the summation of the products of the coördinates of the centers of the segments into which the arch ring is divided, while the denominator is the summation of the products of the ordinates to the random equilibrium polygon taken at the centers of the segments into which the arch ring is divided, and the distances of these ordinates from the abutment A.

Graphic Solution of the Stresses in a Two-hinged Arch.— Divide the given arch ring into a number of segments, varying from 10 to 20 parts, in which $ds/E \cdot I$ equals a constant. Assume that the external loads act through the centers of the segments.. Lay off the loads and construct a force polygon with a pole distance H', and draw the equilibrium polygon so that it will pass through the hinges A and B of the arch. Now to use equation (115′), scale off the ordinate y of each point in the arch ring at which the loads are applied and assume that these ordinates are horizontal loads acting at the points in the arch ring of which they are the ordinates; lay off these ordinates as horizontal loads and with the assumed pole distance H'' draw an equilibrium polygon with the force polygon thus constructed. In like manner assume that the ordinates to the random equilibrium polygon at the corresponding points in the arch are horizontal forces; construct a force polygon with a pole distance H'' (use the same pole distance for both force polygons) and draw an equilibrium polygon with the force polygon thus constructed. The bending moments at the right abutment for the two loadings will be proportional to the horizontal deformation of the hinge B for the two loadings, and r will be equal to the ratio of the two bending moments as given by equation (115′).

To use equation (116′) graphically the loads are taken as acting vertically, and the bending moments at B as calculated by means of the two equilibrium polygons will be proportional to the vertical deforma-

tion at B under the influence of the two systems of loading, and r will be equal to the ratio of the two bending moments.

Having calculated the true pole distance, H, the true equilibrium polygon is drawn as in Fig. 235.

Temperature Stresses.—With an increase or decrease in temperature the arch will expand or contract uniformly and the change in the span will be

$$\Delta x = \pm\, e \cdot t \cdot L \tag{117}$$

where e is the coefficient of linear expansion of the material (e for steel and concrete is approximately 0.0000067 per degree Fahr.); t is the change in temperature in degrees Fahr.; and L is the span of the arch in the same units as Δx.

Then equation (110) becomes

$$\Delta x = \sum \frac{M \cdot y \cdot ds}{E \cdot I} = \pm\, e \cdot t \cdot L \tag{110''}$$

Now if H_t is the horizontal component that will produce the same horizontal movement as the change in temperature as given by (110″) then, $M = H_t \cdot y$, and

$$H_t = \pm \frac{e \cdot t \cdot L}{\sum \frac{y^2 \cdot ds}{E \cdot I}} \tag{118}$$

The total value of the horizontal component of the reaction will be $H + H_t$.

STRESSES IN AN ARCH WITHOUT HINGES. Introduction.—In an arch without hinges the following conditions must be satisfied: (1) the span must be constant; (2) the abutments must maintain the same relative positions; and (3) the tangents to the neutral axis of the arch at the abutments must remain fixed.

From the discussion of the two-hinged arch we have

$$\Delta x = \sum \frac{M \cdot y \cdot ds}{E \cdot I} = 0 \tag{110}$$

$$\Delta y = \sum \frac{M \cdot x \cdot ds}{E \cdot I} = 0 \tag{111}$$

Also from equation (107) $d\phi = M \cdot ds/E \cdot I$, and

$$\Delta \phi = \sum \frac{M \cdot ds}{E \cdot I} = 0 \tag{119}$$

In Fig. 239 at the left abutment, A, the vertical reaction is V_1, the horizontal reaction is H, and the bending moment is $M_1 = H \cdot y_1$.

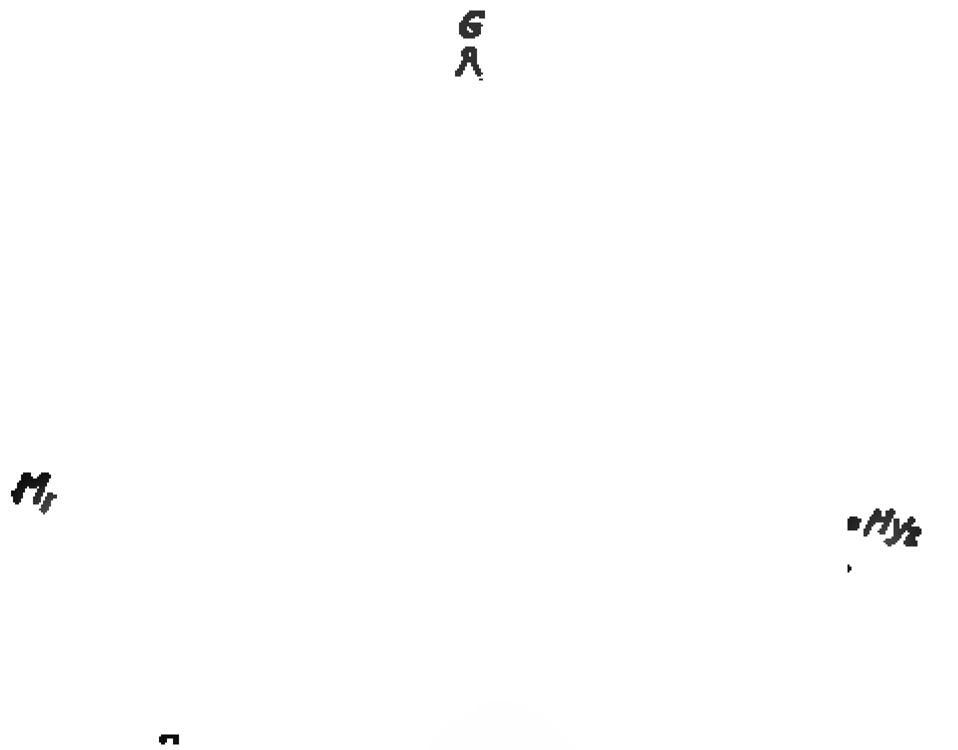


FIG. 239.

At the right abutment, B, the vertical reaction is V_2, the horizontal reaction is H, and the bending moment is $M_2 = H \cdot y_2$. Now if the arch were hinged at A and B, the line of resistance would be the equilibrium polygon AEB drawn with a force polygon having a pole distance equal to the horizontal reaction H for a two-hinged arch. With the arch with fixed ends there is bending moment at points A and B and the equilibrium polygon will pass through points A' and B' and will take the position $A'DB'$, the points a and b being points of contra-flexure (points A' and B' may both be below the abutments, both above the abutments, or one may be below and the other above, depending upon the loading and elastic properties of the arch).

Now if the arch ring is divided into segments so that $ds/E \cdot I = g$, a constant, equations (110), (111) and (119) become

$$\Sigma M \cdot y = 0 \tag{110'}$$

$$\Sigma M \cdot x = 0 \tag{111'}$$

$$\Sigma M = 0 \tag{119'}$$

The bending moment M at the point C will be

$$\begin{aligned} M &= H \cdot t = M' - H(y - FF') \\ &= M' - H[(y - y_1) + x(y_1 - y_2)/L] \end{aligned} \tag{120}$$

where M' is the bending moment due to the vertical loads calculated as in a beam; H is the true horizontal reaction; y is the ordinate of the point in question in the arch referred to the line AB; FF', y_1 and y_2 are as shown; and L is the span of the arch.

Substituting the value of M in (120) in (110′) we have

$$\Sigma M \cdot y = \Sigma M' \cdot y - H \cdot \Sigma[(y - y_1) + x(y_1 - y_2)/L]y = 0$$

and

$$H = \frac{\Sigma M' \cdot y}{\Sigma[y^2 - y \cdot y_1 + x \cdot y(y_1 - y_2)/L]} \tag{121}$$

In like manner by substituting in equation (111′)

$$H = \frac{\Sigma M' \cdot x}{\Sigma[x \cdot y - x \cdot y_1 + x^2(y_1 - y_2)/L]} \tag{122}$$

and by substituting in equation (119′)

$$H = \frac{\Sigma M'}{\Sigma[y - y_1 + x(y_1 - y_2)/L]} \tag{123}$$

From equations (121), (122) and (123) it will be seen that there are three equations and three unknowns, H, y_1 and y_2, so that the problem can be solved by substituting in the three equations of condition. In solving these equations the arch ring must be divided into sections in the same manner as for the graphic solution.

Graphic Solution.—In Fig. 240 ACB is the arch ring and $A'DB'$ is the true equilibrium polygon drawn with a force polygon having a true pole distance, H, the loads being applied at the centers of the

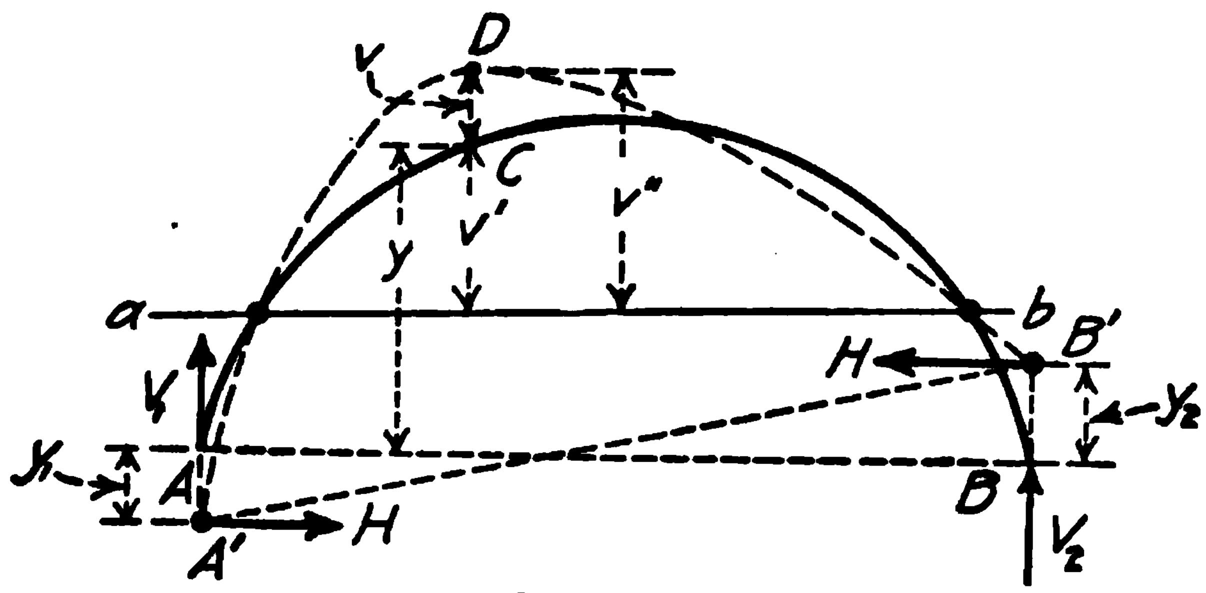


FIG. 240.

segments of the arch determined so that $ds/E \cdot I =$ a constant. Now the bending moment at C will be $M = H \cdot v$, and the equations of condition are found by substituting in (119'), (110') and (111') to be

$$\Sigma v = 0 \tag{124}$$

$$\Sigma v \cdot x = 0 \tag{125}$$

$$\Sigma v \cdot y = 0 \tag{126}$$

Now draw the line ab in Fig. 240 so that it will satisfy the conditions

$$\Sigma v' = 0 \tag{127}$$

$$\Sigma v' \cdot x = 0 \tag{128}$$

in which v' is the vertical distance of any point C from the line ab, x being the abscissa of the point C referred to the abutment A. Now in Fig. 240, $v = v'' - v'$, and substituting in (124)

$$\Sigma v = \Sigma (v'' - v') = \Sigma v'' - \Sigma v' = 0$$

but from (127)

$$\Sigma v' = 0, \text{ and } \Sigma v'' = 0 \tag{129}$$

Substituting in (125)

$$\Sigma v \cdot x = \Sigma (v'' - v')x = \Sigma v'' \cdot x - \Sigma v' \cdot x = 0$$

but from (128)

$$\Sigma v' \cdot x = 0, \text{ and } \Sigma v'' \cdot x = 0 \tag{130}$$

Substituting in (126)

$$\Sigma v \cdot y = \Sigma (v'' - v') y = \Sigma v'' \cdot y - \Sigma v' \cdot y = 0$$

from which

$$\Sigma v'' \cdot y = \Sigma v' \cdot y \tag{131}$$

Equations (129) and (130) may be satisfied by an infinite number of equilibrium polygons drawn for the given loading. In practice assume a pole distance and draw an equilibrium polygon. Draw the closing line ab in Fig. 240 so that $\Sigma v' = 0$, and $\Sigma v' \cdot x = 0$, and then calculate $\Sigma v'' \cdot y$. Also calculate $\Sigma v' \cdot y$ for the arch ring. If H' is the assumed pole distance and H is the true pole distance, then if $\Sigma v'' \cdot y$ is not equal to $\Sigma v' \cdot y$ the true pole distance may be found from the proportion

$$H : H' :: \Sigma v'' \cdot y : \Sigma v' \cdot y$$

Problem 1.—Given a segmental reinforced concrete highway arch having a span of 50′ 0″, a rise of 15′ 0″, and a thickness of 2′ 0″, carrying a spandrel loading as shown and a live load of 400 lbs. per square foot. The solution will be made for a live load over one-half the span. This loading ordinarily gives maximum bending moments in the arch ring. The arch ring will be divided into 10 equal segments, beginning and closing with a half segment as shown; the loads were calculated and numbered 1, 2, 3, 4, etc., as shown in Fig. 241.

Now lay off the loads 1, 2, 3, etc., and construct a force polygon with an assumed pole distance of 28,000 lbs. as in (b). With force polygon in (b) construct equilibrium polygon $V5V'$ in (c). Now draw nn' parallel to VV', nV being made equal to the arithmetical mean of the ordinates to the equilibrium polygon $V5V'$, at the points 1, 2, 3, etc. The sum of the ordinates 1–1′, 2–2′, 3–3′, etc., between nn' and the equilibrium polygon $V5V'$ will now be equal to zero, and the condition $\Sigma v'' = 0$ will be satisfied.

To satisfy equation (130), $\Sigma v'' \cdot x = 0$, the center of gravity of

the positive ordinates at 1, 2, 3, etc., to the equilibrium polygon, $V5V'$ must be in the same vertical line with the center of gravity of the negative ordinates 1, 2, 3, etc., of the negative polygon $Vnn'V'$.

To calculate the center of gravity of the ordinates to the equilibrium polygon $V5V'$, lay off the loads 1, 2, 3, etc., in (d) proportional to the ordinates 1, 2, 3, etc., to the equilibrium polygon $V5V'$, assume a pole distance (for convenience the pole distance was taken the same as in (b)), and draw the equilibrium polygon in (e). The line $O''b$ in (d) drawn parallel to the closing line ac in (e) gives the reactions R_1 and R_2. Now for a symmetrical arch we have with sufficient accuracy

$$mV = 2(2R_1 - R_2)/n, \text{ and } m'V' = 2(2R_2 - R_1)/n$$

Then

$$mV : nV :: 2R_1 - R_2 : \tfrac{1}{2}(R_1 + R_2)$$

and

$$m'V' : n'V' :: 2R_2 - R_1 : \tfrac{1}{2}(R_1 + R_2)$$

(These relations are exact for equally spaced ordinates, and are sufficiently exact for all ordinary cases.)

Lay off $m'V'$ and mV and draw mm'. The equation $\Sigma v'' \cdot x = 0$ is now satisfied.

Equations (129) and (130) are now satisfied and intercepts 1–1′, 2–2′, 3–3′, etc., are proportional to the true moments in the arch. To determine the true pole distance equation (131) must be satisfied. This requires that $\Sigma v' \cdot y = \Sigma v'' \cdot y$.

In (a) draw line ab parallel to AB, by making aA equal to the arithmetical mean of the ordinates to the arch ring at the centers of the segments 1, 2, 3, etc. Then if the intercepts 1–1′, 2–2′, 3–3′, etc., are considered as horizontal loads, $\Sigma v' \cdot y$ may be calculated as follows: In (f) lay off the ordinates 1–1′, 2–2′, 3–3′, etc., to the arch ring as horizontal loads acting at points in the arch ring (the loads in (f) are taken twice their true value), assume a pole distance, in this case $H = 12$ feet, and draw equilibrium polygon (f′). In (g) lay off ordinates 1–1′, 2–2′, etc., in equilibrium polygon (c) as horizontal loads

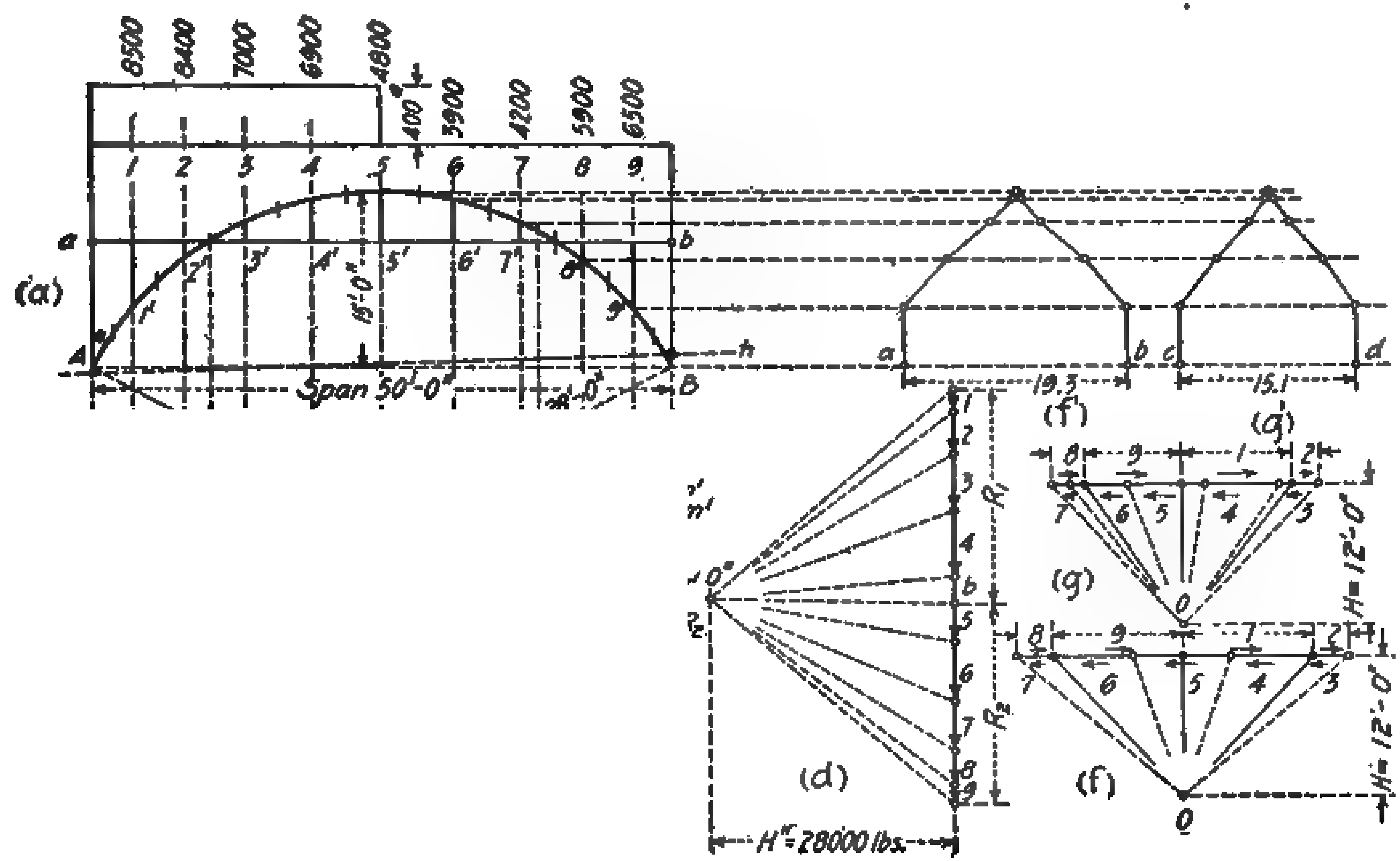

8500
8400
7000
6900
4800
400
3900
4200
5900
6500
(a)
Span 50'-0"
15'-0"
H" = 28000 lbs.
(d)
R1
R2
(f')
(g')
19.3
15.1
(g)
(f)
H = 12'-0"

Fig. 241.

acting at corresponding points in the arch ring, assume a pole distance, for convenience the same pole distance should be assumed as in (f), and draw equilibrium polygon (g′). The true pole distance will then be $H = 15.1/19.5 \times 28{,}000 = 21{,}900$ lbs.

To determine the pole O' in (b) Fig. 241 draw Ob parallel to VV' in (c), through b in (b) draw bO'' parallel to the line gh in (a) which is located by laying off $ag = mV \times 28{,}000/21{,}900$ and $bh = m'V' \times 28{,}000/21{,}900$. The pole O'' will be on the line bO'' at a distance $H = 21{,}900$ lbs. from the load line.

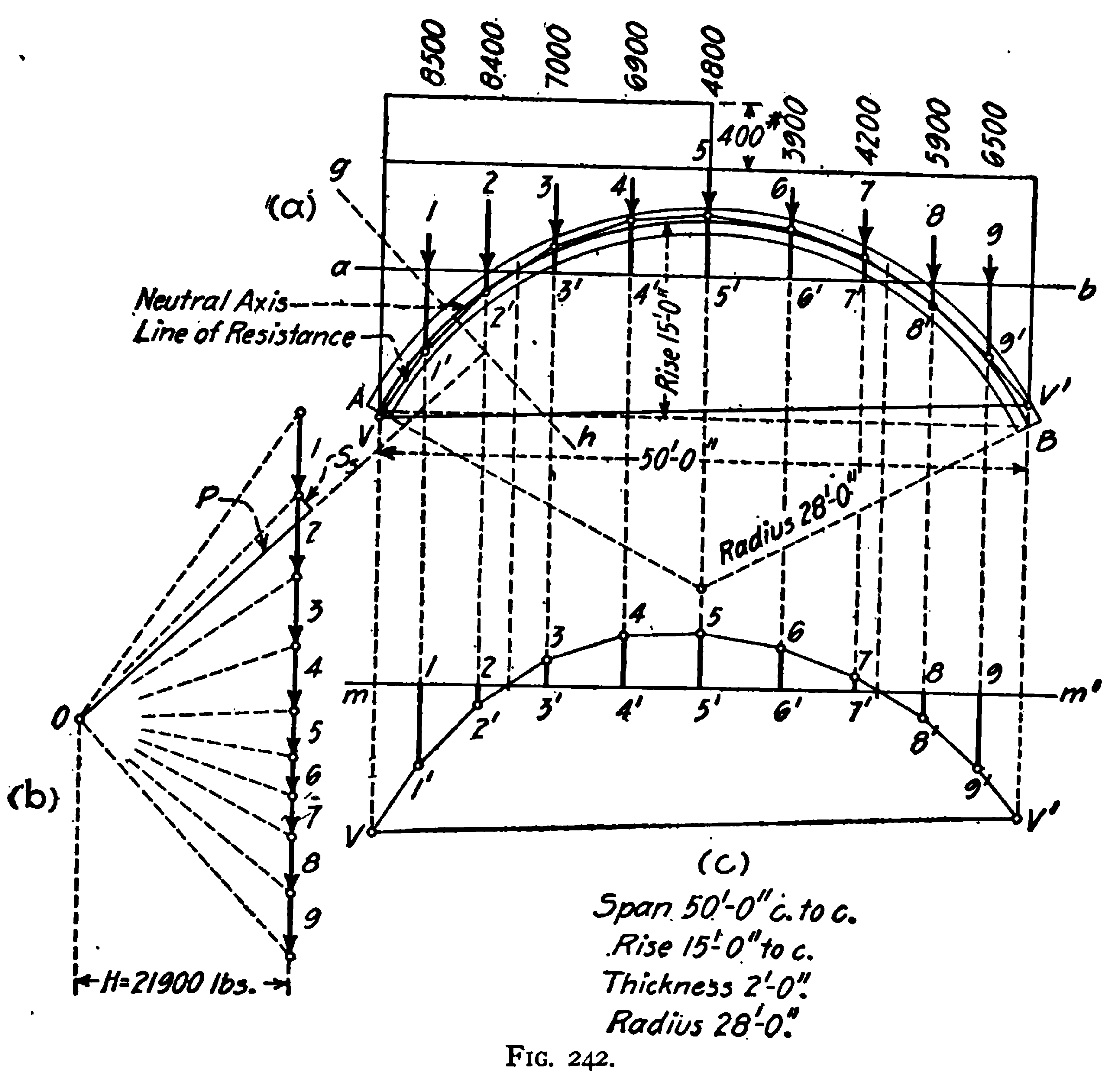


FIG. 242.

To draw the true equilibrium polygon in Fig. 242 lay off the loads 1, 2, 3, etc., and take the pole distance $H = 21{,}900$ lbs. as in (b) Fig.

241 and (b) Fig. 242. In (a) Fig. 242 lay off $aV = ag$ in (a) Fig. 241 $= mV$ in (c) Fig. 242, and lay off $bV' = bh$ in (a) Fig. 241 $= m'V'$ in (c) in Fig. 242. Now beginning at V in (a) with the force polygon in (b) construct the true equilibrium polygon, the line VV' being the closing line.

The shear at the section $gh = 2{,}000$ lbs. The direct thrust in the arch ring at the section gh is $P = 32{,}000$ lbs. The eccentricity of P at the section gh is $t = 4''$, and the bending moment will be $M = 32{,}000 \times 4 = 128{,}000$ in.-lbs.

The maximum stress on the section gh is

$$S = P/A \pm M \cdot c/I$$

$$S = \frac{32{,}000}{24 \times 12} \pm \frac{128{,}000 \times 12}{\frac{1}{12} \times 12 \times 24^3}$$

$$= +111 \pm 111 = +222 \text{ or } 0 \text{ lbs.}$$

For the design of a reinforced concrete arch ring, see Chapter XVII.

Temperature Stresses.—With an increase or decrease in temperature the arch will expand or contract uniformly if there is no resistance. The tangents at the abutments will remain fixed which requires that $\Sigma(M \cdot ds/E \cdot I) = 0$. The abutments will remain at the same relative heights, which requires that $\Sigma(M \cdot x \cdot ds/E \cdot I) = 0$. Now if there be no constraint and the arch is free to move under the load

$$\Delta x = \sum \frac{M \cdot y \cdot ds}{E \cdot I} = e \cdot t \cdot L \tag{132}$$

Now if the movement in Fig. 243 is prevented by the horizontal reaction H', the value of M in (132) will be $= H' \cdot v'$, where v' has values as will now be described.

In Fig. 243 draw the line ab so that $\Sigma(v' \cdot ds/E \cdot I) = 0$, where v' is the distance from the line ab to the arch ring. Now the horizontal reaction H' must be sufficient to bring the arch back to its original position. Substituting in (109)

$$\Delta x = \sum \frac{M \cdot y \cdot ds}{E \cdot I} = \frac{\Sigma H' \cdot v' \cdot y \cdot ds}{E \cdot I} \tag{133}$$

Now make $ds/E \cdot I = g$, a constant, and

$$\Delta x = H' \cdot g \Sigma v' \cdot y$$

and

$$H' \cdot g \Sigma v' \cdot y = e \cdot t \cdot L$$

$$H' = \pm e \cdot t \cdot L / (g \cdot \Sigma v' \cdot y)$$

Now for a change of 50° Fahr.

$$e \cdot t \cdot L = 0.0000067 \times 50 \times 600'' = 0.20''$$

$$g = ds/E \cdot I = (6 \times 12)/(2{,}000{,}000 \times 13{,}824) = 1/384{,}000{,}000$$

$$\Sigma v' \cdot y = \tfrac{1}{2} \times 19.3 \times 12 \times 144 = 16{,}680$$

$$H = \pm \frac{0.20}{\frac{1}{384{,}000{,}000} \times 16{,}680} = \pm 4{,}600 \text{ lbs.}$$

Stresses Due to Rib Shortening.—The direct thrust will cause a shortening of the arch rib in addition to the stresses already calcu-

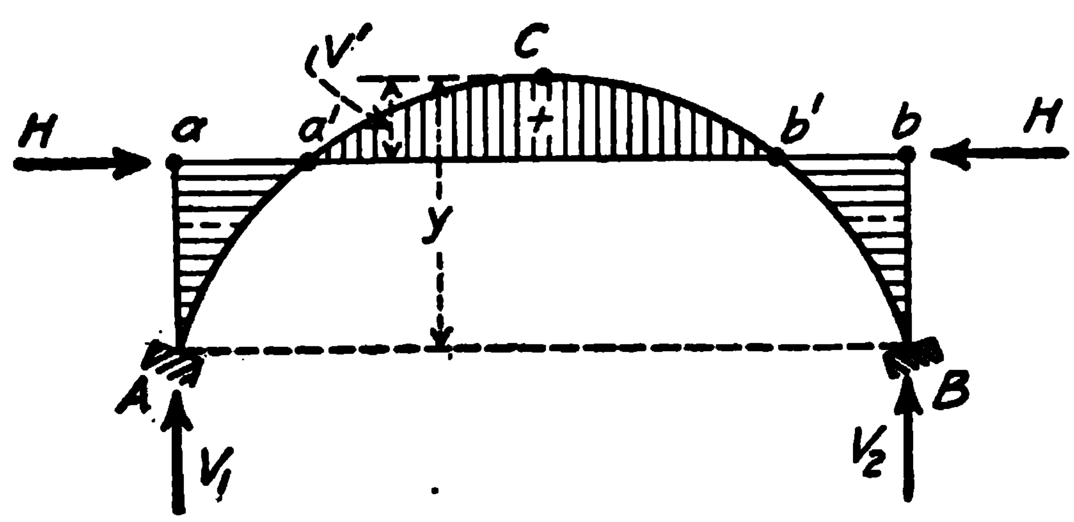


FIG. 243.

lated. If the direct thrust were constant for all sections of the arch ring the effect would be approximately the same as a decrease in temperature. Now if e is the coefficient of linear expansion per degree Fahr., the change in temperature that will have the same shortening effect as the direct thrust will be

$$t = P/(A \cdot E \cdot e)$$

where P is the direct thrust, A is the area of the section, and E is the modulus of elasticity.

In this arch

$$t = 32{,}000/(12 \times 24 \times 2{,}000{,}000 \times 0.0000067) = 8.4°$$

The value of H for this decrease in temperature will be $8.4/50 \times 4{,}600$ lbs. $= -750$ lbs.

Summary.—To provide for the stresses due to direct loads, the temperature and the direct thrust the pole distance will be

$$H = +21{,}900 \pm 4{,}600 \mp 750 = 25{,}750 \text{ or } 16{,}350 \text{ lbs.}$$

To complete the analysis of the stresses in the arch, the stresses should be calculated in the same manner as in Fig. 242 for a horizontal thrust of 25,750 lbs. and 16,350 lbs.

For further information on the calculation of stresses in solid arches, see Howe's "Symmetrical Masonry Arches," Turneaure and Maurer's "Principles of Reinforced Concrete," and Buel and Hill's "Reinforced Concrete."

CHAPTER XVII.

Design of Masonry Bridges and Culverts.

Introduction.—When properly constructed, concrete or stone masonry bridges are permanent, and this type of structure should be given the preference when conditions are favorable for its construction. Masonry bridges may be classed under (1) arches, (2) beam or truss leg spans, (3) culverts. Masonry arch highway bridges may be built of concrete or stone masonry. The discussion in this chapter will be confined chiefly to concrete construction. The principles deduced for concrete arches may be used in the design of stone masonry arches. Before taking up the design of concrete structures it will be necessary to discuss very briefly the theory of reinforced concrete. For a more complete discussion of reinforced concrete the reader is referred to "Principles of Reinforced Concrete Construction" by Turneaure and Maurer, published by John Wiley & Sons.

Theory of Reinforced Concrete.

CONCRETE.—Concrete is fabricated by thoroughly mixing hydraulic cement, sand and gravel or broken stone, together with sufficient water to produce a mixture of the proper consistency. The mixture is immediately deposited in molds and is well tamped. When concrete is reinforced with steel it is mixed with more water than when it is used plain. For standard specifications for concrete, see Chapter XV.

Strength.—The strength of concrete varies with the proportions of the mixture, the quality of the cement and other ingredients, the method of mixing and depositing and the age. The following may be taken as average values. The cement is assumed to be a first-class American Portland, the sand a clean, moist, bank sand, and the stone is equal in quality to a first-class limestone and has about 45 per cent voids. The proportions were determined by volume, the cement being packed in the barrel or sack. Gravel concrete may be taken as from 75 to 80 per cent of the strength of broken stone concrete.

Compressive Strength.—Average values for the compressive strength of first-class Portland cement concrete of different proportions are given in Table LVI.

TABLE LVI.

COMPRESSIVE STRENGTH OF PORTLAND CEMENT CONCRETE IN POUNDS PER SQUARE INCH.

PROPORTIONS.	AGE, 1 MONTH.	AGE, 6 MONTHS.
1 cement, 2 sand, 4 broken stone	2,440	3,300
1 cement, 2 sand, 5 broken stone	2,350	3,180
1 cement, 3 sand, 5 broken stone	2,030	2,740
1 cement, 3 sand, 6 broken stone	1,950	2,630
1 cement, 3 sand, 8 broken stone	1,800	2,400
1 cement, 4 sand, 8 broken stone	1,570	2,120

Tensile Strength.—The tensile strength is difficult to determine by direct experiment, and is usually taken as one-tenth of the compressive strength. With factor of safety of 4 to 6 the safe tensile stresses will vary from 80 to 40 lbs. per sq. in.

Working Stresses.—To obtain safe working stresses use a factor of safety of from 4 to 6. Working stresses of 500 to 600 lbs. per sq. in. for compression, and 50 lbs. per sq. in. for tension are very commonly used for good 1–3–6 Portland cement concrete (tension is neglected in the calculation of reinforced concrete beams).

Modulus of Elasticity.—The modulus of elasticity of concrete is not a constant quantity as is the modulus of elasticity of steel, but varies with the proportions and ingredients, and with the stress in the concrete. The following values have been proposed for use in reinforced concrete design where wet Portland cement concrete is used.

TABLE LVII.

MODULUS OF ELASTICITY OF PORTLAND CEMENT CONCRETE.

PROPORTIONS.		MODULUS OF ELASTICITY, POUNDS PER SQUARE INCH.
	1 : 1½ : 3	4,000,000
	1 : 2 : 4	3,000,000
Broken stone or gravel concretes	1 : 2 : 5	2,500,000
	1 : 3 : 6	2,000,000
	1 : 4 : 8	1,500,000
Cinder concrete	1 : 2 : 5	508,000

Shear.—Very few experiments have been made to determine the shearing strength of concrete. Recent tests would appear to show that the shearing strength of concrete is from 60 to 80 per cent of the compressive strength.

Adhesion to Steel Rods.—Where the yield point of the rod is not exceeded the adhesion of smooth rods to concrete varies from about 275 to 700 pounds per square inch of surface in contact. Deformed rods have a greater adhesion than plain rods. With a factor of safety of 6 to 4, the safe adhesion will vary from 50 to 80 lbs. per sq. in. for 1–3–6 Portland cement concrete. Experience would show that deformed rods should be imbedded in concrete not less than 30 diameters and plain rods not less than 50 diameters in order that the tensile strength of the rod be developed.

Tests of Reinforced Concrete Beams.—Many tests have been made to determine the strength and behavior of reinforced concrete beams. The tests by Professor A. N. Talbot, recorded in University of Illinois Engineering Experiment Station Bulletin, Nos. 1, 4, 8, 10, 12 and 14, are very valuable and instructive.*

RESISTANCE OF REINFORCED CONCRETE BEAMS TO FLEXURE.—Reinforced concrete beams may fail by (1) tension of steel; (2) compression of concrete; (3) shear of concrete; (4) bond or slip of rods; (5) diagonal tension of concrete; (6) miscellaneous methods, like the splitting of concrete away from the rods, crushing of bearings, etc. A given beam is not liable to fail by all the methods.

A careful study of the experiments of Professor Talbot and others, shows that the third or straight line stage referred to by Professor Talbot gives the most satisfactory formulas for use in design.

This theory assumes the following principles:

1. A plane section before bending remains a plane section after bending.
2. Tension is carried entirely by the steel.
3. No initial tension or compression in the steel.
4. Perfect adhesion of concrete to the steel.
5. Constant modulus of elasticity of concrete, as in the case of simple beams of steel.
6. The unit deformation in any horizontal fiber varies directly as its distance from the neutral axis.
7. In any cross-section of a beam the sum of the tensile stresses equals the sum of the compressive stresses.

*A summary of Professor Talbot's conclusions are given in the author's book "The Design of Walls, Bins and Grain Elevators," Chapter IV. Copies of these and other bulletins may be obtained by addressing the director of the Station.

8. The moment of the stresses in any section must equal the moment of the applied loads on either side of the section.

The maximum stress in the concrete is about 15 per cent less when calculated by the parabolic theory proposed by Professor Talbot than by the straight line theory given below.

Notation.*—The following notation will be used:

b = breadth of rectangular beam.

d = distance from the compression face to the center of the metal reinforcement.

d' = distance from the center of the reinforcement to center of gravity of compressive stresses.

k = ratio of distance between compression face and neutral axis to distance d.

z = distance from compression face to center of gravity of compressive stresses.

A = area of cross-section of metal reinforcement.

$p = A/b \cdot d$ = ratio of area of metal reinforcement to area of concrete above center of reinforcement.

o = circumference or periphery of one reinforcing bar.

m = number of reinforcing bars.

E_s = modulus of elasticity of steel.

E_c = modulus of elasticity of concrete in compression.

$n = E_s/E_c$ = ratio of two moduli.

f = tensile stress per unit of area in the metal reinforcement.

c = compressive stress per unit of area in most remote fiber of concrete.

ϵ_s = deformation per unit of length in the metal reinforcement.

ϵ_c = deformation per unit of length in most remote fiber of the concrete.

M = bending moment at the given section per inch of width.

M' = total bending moment at the given section.

s = horizontal tensile stress per unit of area in the concrete.

t = diagonal tensile stress per unit of area in the concrete.

u = bond stress per unit of area on the surface of the reinforcing bars.

v = vertical shearing stress and horizontal shearing stress per unit of area in the concrete.

* Notation adopted by Professor Talbot in University of Illinois Engineering Experiment Station Bulletin, No. 4.

Distribution of Stresses in Beams.—In (c) Fig. 244, b is the breadth, d is the depth of the beam above the center of the steel, and $k \cdot d$ is the distance of the neutral axis below the top of the beam, k being a ratio.

In (a) the deformations are shown to be proportional to the distance from the neutral axis, and in (b) the stress in the steel is n times the stress in concrete at the same distance from the neutral axis.

Now in (b) Fig. 244 we have summation of horizontal compressive stresses equal to horizontal tensile stress in steel, and

$$\tfrac{1}{2}c \cdot k \cdot d = f \cdot A/b = f \cdot p \cdot d \qquad (136)$$

since

$$p = A/b \cdot d$$

and

$$\tfrac{1}{2}c \cdot k = f \cdot p \qquad (137)$$

We also have from (b) that

$$c : f/n :: k \cdot d : d(1 - k)$$

$$f \cdot k \cdot d = c \cdot n \cdot d(1 - k)$$

$$f \cdot k = c \cdot n(1 - k) \qquad (138)$$

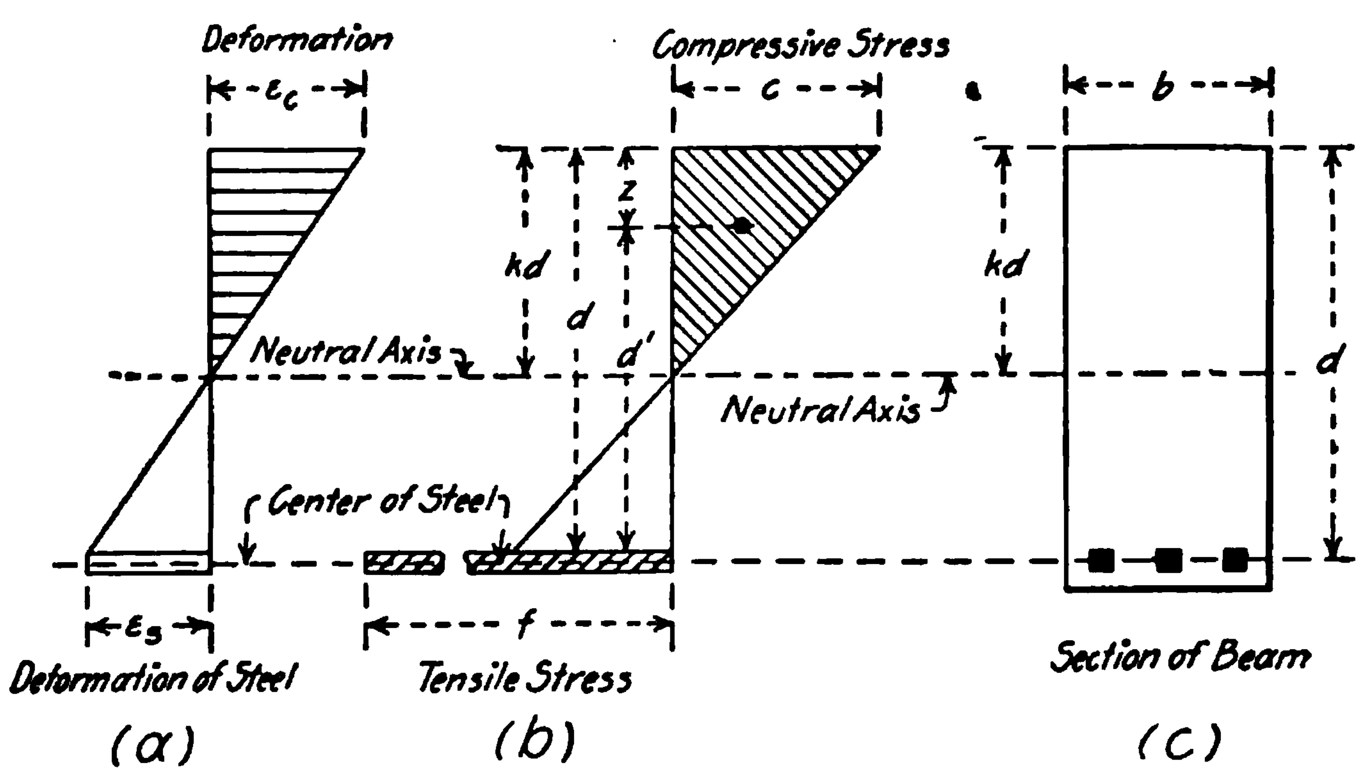


FIG. 244.

And substituting the value of f from (137)

$$\tfrac{1}{2}c \cdot k^2 = c \cdot p \cdot n(1 - k)$$
$$\tfrac{1}{2}k^2 = p \cdot n(1 - k)$$
$$k = -p \cdot n + \sqrt{2p \cdot n + p^2 \cdot n^2} \tag{139}$$

Since the bending moment equals the resisting moment, if M = the bending moment per inch of width, then

$$M = \tfrac{1}{2}c \cdot k \cdot d \cdot d' = A \cdot f \cdot d'/b \tag{140}$$
$$= \tfrac{1}{2}c \cdot k \cdot d^2(1 - \tfrac{1}{3}k) = A \cdot f \cdot d(1 - \tfrac{1}{3}k)/b \tag{141}$$

Hence the formula for the strength of a rectangular beam per inch of width in terms of strength of steel, f, is

$$M = A \cdot f \cdot d(1 - \tfrac{1}{3}k)/b \tag{142}$$

And in terms of compresive strength of concrete, c,

$$M = \tfrac{1}{2}c \cdot k \cdot d^2(1 - \tfrac{1}{3}k) \tag{143}$$

In Fig. 245 values of k have been plotted for values of p from 0.002 (0.2 per cent) to 0.024 (2.4 per cent), and for $n = 10$, $n = 15$, and $n = 20$.

The allowable bending moments for $f = 10{,}000$ lbs. per sq. in.* for different values of p and n are plotted for concrete beams 12 inches wide and from 4 to 36 inches deep, varying by 2 inches in depth. Corresponding values of the compression in the concrete are given on the right of the diagram. The use of the diagram will be illustrated by solving two problems:

Problem 1.—Required the safe uniform load including the weight of the beam for a reinforced concrete beam 12 inches wide and 24 inches deep to the center of reinforcement, and 20 feet long, for $f = 10{,}000$ lbs. per sq. in., $p = 0.01$ (1 per cent), concrete 1–3–6.

Solution.—For 1–3–6 concrete $E_c = 2{,}000{,}000$ and $n = 15$. Entering the right hand table on the lower edge with $p = 0.01$ pass up to the curve $n = 15$, and on the right edge the stress in concrete, c, equals 480 lbs., and on the left edge $k = 0.42$ and $(1 - \tfrac{1}{3}k) = 0.86$. Then on horizontal line $(1 - \tfrac{1}{3}k) = 0.86$ pass to the left until it intersects the inclined line "Beam 24″," then pass down on the vertical line passing through the intersection of the horizontal line and the "Beam" line

* Factor of safety of 6 in the steel.

until the inclined line $p = 0.01$ is cut; then on the horizontal line passing through the intersection of the vertical line and the line of $p = 0.01$ pass to the left edge where we find

$$M' = 50{,}000 \text{ ft.-lbs.}$$

To find the safe uniform load we have

$$M' = \tfrac{1}{8} W \cdot L,$$

and

$$W = 20{,}000 \text{ lbs.},$$

or

$$w = 1{,}000 \text{ lbs. per lineal foot.}$$

The area of steel reinforcement, A, is given by the intersection of the line $p = 0.01$ and " Beam " line and is a little less than 3 sq. in.

The entire depth of beam would be 26", and the safe load per lineal foot exclusive of weight of beam would be $w' = w - 325 = 675$ lbs.

If p were taken equal to 0.02 we would have $c = 750$ lbs.; $k = 0.535$; $M' = 95{,}000$ ft.-lbs.; $w = 1{,}900$ lbs.; and $w' = 1{,}575$ lbs. Which gives a much greater safe load but a much larger stress in the concrete than is usually allowed.

For a fiber stress in the steel $f = 15{,}000$ lbs. per sq. in. in the problem above, add 50 per cent to the stress in the concrete on the right and 50 per cent to the allowable bending moment on the left.

Problem 2.—Required the depth of a reinforced concrete beam 12 inches wide that will carry a bending moment of 80,000 ft.-lbs., $f = 10{,}000$ lbs. per sq. in.; $p = 0.02$ (2 per cent), using 1-2-4 concrete.

Solution.—For 1-2-4 concrete $E_c = 3{,}000{,}000$ and $n = 10$. Now enter the table on the right with $p = 0.02$ and pass up a vertical line to curved line $n = 10$; then on the right $c = 870$ lbs.; and on the left $k = 0.462$. Following across the second diagram to the left as in Problem 1, we see that a beam 22 inches deep will carry 82,000 ft.-lbs., which is sufficient.

This beam is designed for a factor of safety of about 4 for the concrete and about 6 for the steel. The reinforcement should be $p = 0.01$, for which a 30" beam is required. In this beam $c = 555$ lbs.

Ratio of Reinforcement and Working Stresses.—From the diagram in Fig. 245 it will be seen that to obtain the same factor of safety in both concrete and steel the reinforcement should be as follows: For

1–2–4 concrete, $p=0.01$ (1 per cent); for 1–3–6 concrete, $p=0.009$ (0.9 per cent); for 1–4–8 concrete, $p=0.007$ (0.7 per cent). Excessive reinforcement results either in a waste of steel or in excessively high compressive stresses in the concrete.

Bond or Resistance to Slipping of Reinforcing Bars.—Where there is no web reinforcement the shear is taken by the concrete and the shear increments are transferred to the bars by the adhesion of the concrete to the bars. The solution is the same as that for finding the pitch of rivets in the flanges of a plate girder.

Now in (b) Fig. 244 take two right sections at a distance dx apart. Equilibrium of these two sections is maintained by the resisting moment of the bond which is equal and opposite to the moment of the vertical shear, a couple with an arm dx.

Taking moments about center of gravity of compressive forces we have

$$V \cdot dx = m \cdot o \cdot u \cdot dx \cdot d' \qquad (144)$$

where m = number of bars, o = surface of bar for one inch in length, u = bond developed per square inch of surface of bar, and V is the vertical shear in the beam.

Solving for u, we have

$$u = V/(m \cdot o \cdot d') \qquad (145)$$

Equation (145) applies to the case of horizontal bars. For inclined bars d' will be a variable and u will be the horizontal component of the bond resistance.

Vertical and Horizontal Shearing Stresses.—At any point in a beam the vertical unit shearing stress is equal to the horizontal unit shearing stress. The horizontal shearing stress transmits the increments of tension to the reinforcing bars by bond stresses, as explained in the preceding discussion.

The amount of this horizontal stress transmitted to the reinforcing bars is by equation (145)

$$m \cdot o \cdot u = V/d'$$

Now if the horizontal shear just above the plane of the bars is v, the total horizontal shearing stress will be $v \cdot b$, and

$$v = V/b \cdot d' \qquad (146)$$

As no tension is assumed to exist in the concrete, the horizontal shear

will be constant up to the neutral axis, above which point it decreases to zero at the top of the beam. It will be seen that lean or poor concrete lacking in shearing strength should not be placed below the neutral axis of beams with the idea that it may be satisfactory for the reason that the concrete is assumed to take no tension.

Diagonal Tension in Concrete.—In Mechanics of Materials (Merriman's Mechanics of Materials, p. 265, 1905 edition) it is shown that shear and tensile stresses combine to cause diagonal tensile stresses

$$t = \tfrac{1}{2}s + \sqrt{\tfrac{1}{4}s^2 + v^2} \qquad (147)$$

where t is the diagonal tensile unit stress, s is the horizontal tensile unit stress, and v is the horizontal or vertical shearing unit stress. The direction that stress t makes with the horizontal is one-half the angle whose cotangent is $\frac{1}{2}s/v$. If there is no tension in the concrete this reduces to

$$t = v \qquad (148)$$

and t makes an angle of 45° with the horizontal.

Stresses due to diagonal tension may be carried (1) by bending the reinforcing bars, or strips sheared from them, into a diagonal position, or (2) by means of stirrups to take the vertical component of the diagonal tension.

Depth of Concrete Below Bars.—For adequate fire protection a thickness of 2 inches is necessary. Where fire protection is not required Edwin Thacher, M. Am. Soc. C. E., uses a thickness of 1″ for beams 4 inches deep, 2″ for beams 20 inches and over, and proportional thickness for the depths between 4 and 20 inches.

Spacing of Bars.—An arbitrary rule is to space bars no nearer together in the clear than two diameters, and in no case less than 1½ inches apart, nor nearer than 1½ inches to either side of the beam.

Length of Bar to Imbed in Concrete.—Experiments show that plain bars should be imbedded for a length of not less than 50 diameters, and deformed bars for not less than 30 diameters. Where bars can not be imbedded for this length they should be bent or anchored.

Shrinkage and Temperature Stresses.—The coefficient of expansion of concrete is practically the same as for steel, and is about ± 0.0000065. The shrinkage in hardening causes considerable stress, but this stress is not unduly high unless the ratio of the steel is high.

If the structure is not free to expand or contract cracks will form at intervals. If plain concrete expands and contracts the same as reinforced concrete, as is usually assumed, it will not be possible to prevent cracks. The reinforcement, however, will cause the cracks to take place so close together that they will not be large, and may be invisible.

The cracks will not become large until after the steel has passed its elastic limit. The temperature stress in the steel must be considered. The percentage of reinforcement, p, required to prevent large cracks in concrete will be

$$p = \frac{\text{tensile strength of concrete}}{\text{elastic limit of steel minus temperature stress in steel}}$$

For a change of 50° the temperature stress in the steel $= 50 \times .0000065 \times 30{,}000{,}000 = 9{,}750$ lbs. per sq. in. For a tensile stress of 200 lbs. per sq. in. in the concrete, and an elastic limit of 40,000 lbs. per sq. in. in the steel

$$p = 200/30{,}250 = .0066$$

For an elastic limit of 60,000 lbs. per sq. in. in the steel

$$p = 200/50{,}250 = .004$$

It will be seen that for reinforcement against temperature stresses a high steel with mechanical bond is desirable.

Stresses in T-beams.—The following additional notation will be used:

$b =$ width of flange.

$b' =$ width of web.

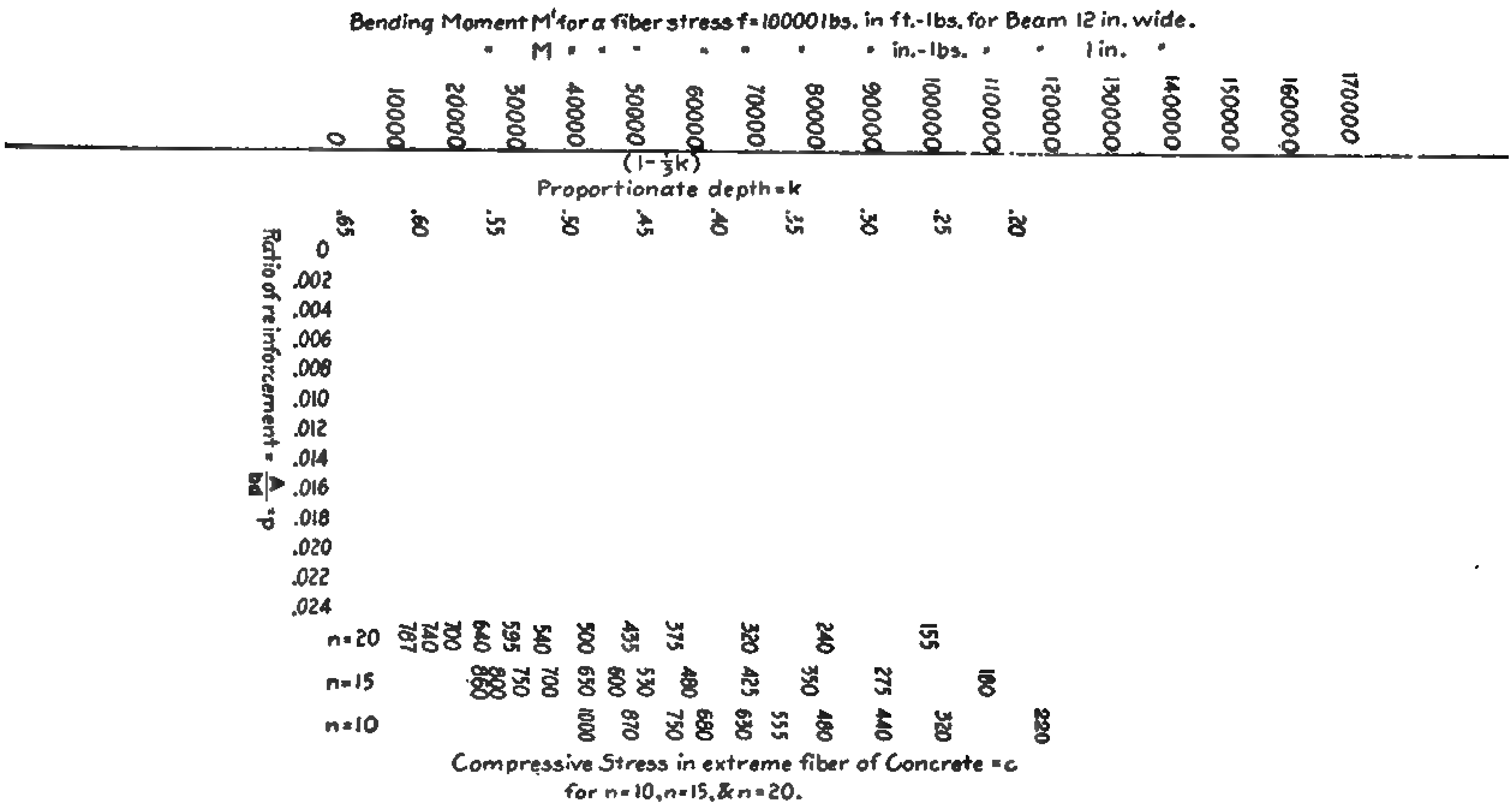
$d =$ depth from top of flange to center of steel reinforcement.

$t =$ thickness or depth of flange.

There are two cases to consider: (1) where the neutral axis is in the flange, and (2) where the neutral axis is in the web.

(1) *Neutral Axis in the Flange.*—Formulas (136) to (143), inclusive, apply to this case, b in the formulas representing flange width, and $p = A/b \cdot d$. T-beams for case (1) may be designed by diagram, Fig. 245, the b and d used being the same as for rectangular beams.

The arm of the couple, consisting of the compressive and tensile stresses, will always be greater than $d' = d - \frac{1}{3}t$, and the following approximate formula for tension in the steel is on the safe side



Bending Moment M' for a fiber stress f = 10000 lbs. in ft.-lbs. for Beam 12 in. wide.
" M " " " " " " in.-lbs. " " 1 in. "
0
10000
20000
30000
40000
50000
60000
70000
80000
90000
100000
110000
120000
130000
140000
150000
160000
170000
(1 - ⅓k)
Proportionate depth = k
.65 .60 .55 .50 .45 .40 .35 .30 .25 .20
Ratio of reinforcement = A/bd = p
0 .002 .004 .006 .008 .010 .012 .014 .016 .018 .020 .022 .024
n = 20 787 740 700 640 595 540 500 435 375 320 240 155
n = 15 860 800 750 700 650 600 530 480 425 350 275 180
n = 10 1000 870 750 680 630 555 480 440 320 220
Compressive Stress in extreme fiber of Concrete = c
for n = 10, n = 15, & n = 20.

$$M' = A \cdot f(d - \tfrac{1}{3}t) \tag{149}$$

where M' = total bending moment in the beam.

(2) *Neutral Axis in the Web.*—The amount of the compression in the web is small as compared with that in the flange and will be neglected. The arm of the couple, consisting of the compressive and tensile stresses, will never be as small as $d' = d - \frac{1}{2}t$, while the average compressive stress will never be as small as $\frac{1}{2}c$, except when the neutral axis is at the top of the web. The approximate formulas for case (2) are

$$M' = A \cdot f(d - \tfrac{1}{2}t) \tag{150}$$

$$M' = \tfrac{1}{2}c \cdot b \cdot t(d - \tfrac{1}{2}t) \tag{151}$$

Shear.—The shear in a T-beam will be practically the same as in a rectangular beam having a width b', equal to the thickness of the web.

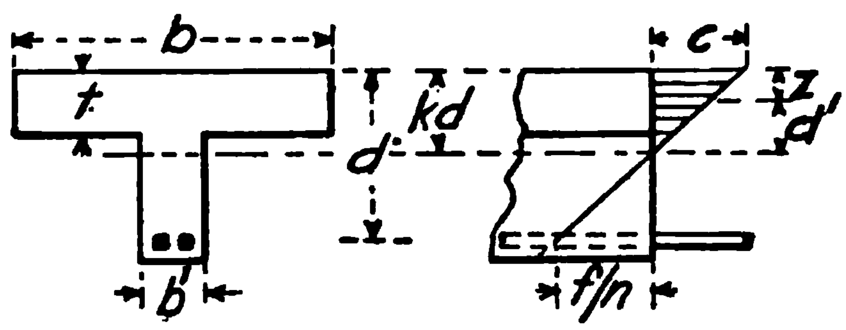


FIG. 246.

Stresses in Beams with Double Reinforcement.—The following additional notation will be used:

A' = area of the cross-section of the metal in the compressive reinforcement.

$p' = A'/b \cdot d$ = ratio of steel in compression to the area of concrete above the tensile reinforcement.

f' = compressive stress in steel reinforcement.

z' = distance from the top of the beam to the resultant of the compressive forces in the beam.

d'' = distance from the top of the beam to the center of the steel in compression.

Now from (138)

$$f = n \cdot c \cdot (1 - k)/k \tag{138}$$

and also

$$f' = n \cdot c \cdot (k - d''/d)/k \tag{152}$$

Now the compressive stresses are equal to the tensile stresses and

$$f \cdot A = \tfrac{1}{2} c \cdot k \cdot b \cdot d + f' \cdot A' \tag{153}$$

Now inserting values of f and f' from (138) and (152) the location of the neutral axis may be obtained from the formula

$$k^2 + 2n(p + p')k = 2n(p + p' \cdot d''/d) \tag{154}$$

Now

$$d' = d - z$$

For approximate calculations we may take $d' = 0.85d$, $k = 0.45$, and

$$M' = 0.85\, p \cdot f \cdot b \cdot d^2 \tag{155}$$

$$M' = (0.19 + 10.5p')c \cdot b \cdot d^2 \tag{156}$$

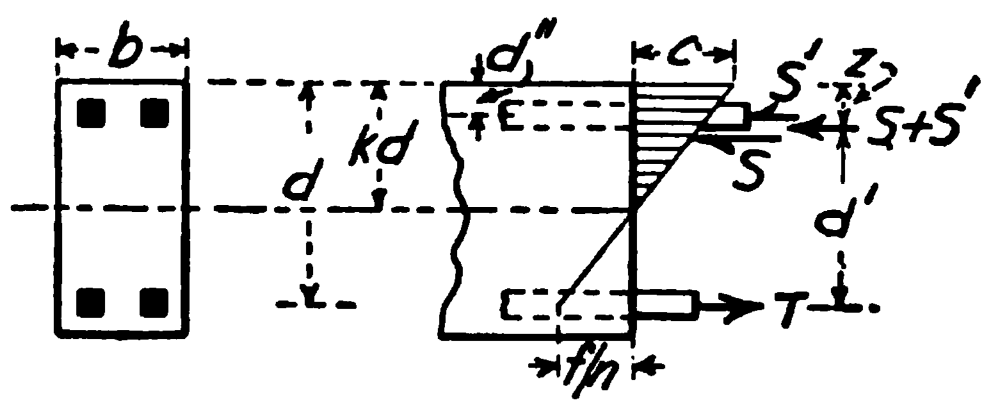


FIG. 247.

Flexure and Direct Stress.—When the normal resultant of the forces acting on the section of a beam does not act through the neutral axis of the section there will be both flexural and direct stresses.

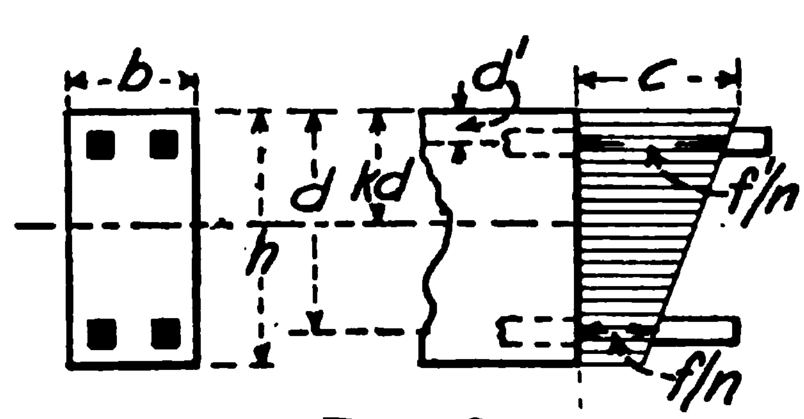


FIG. 248.

In concrete beams the direct stress is always compression, and there are therefore two cases: (1) all compressive stresses, and (2) part compressive and part tensile stresses.

(1) *Stresses all Compressive.*—The maximum compression on the concrete will be

$$c = \frac{N}{b \cdot h + A \cdot n + A' \cdot n} + \frac{M' \cdot k \cdot d}{I_c + n \cdot I_s} \tag{157}$$

and the maximum compression in the steel on the side in which the concrete is stressed highest is

$$f' = \frac{n \cdot N}{b \cdot h + A \cdot n + A' \cdot n} + \frac{n \cdot M'(k \cdot d - d')}{I_c + n \cdot I_s} \tag{158}$$

and the maximum tension in the steel on the other side is

$$f = \frac{n \cdot N}{b \cdot h + A \cdot n + A' \cdot n} - \frac{n \cdot M'(d - k \cdot d)}{I_c + n \cdot I_s} \tag{159}$$

where

N = the direct normal stress.

A' = the area of the steel on the side of the beam having maximum compression.

A = the area of the steel on the other side.

h = entire depth of beam.

I_c = moment of inertia of the concrete about the neutral axis of the transformed beam; if the reinforcement is symmetrical about the center of the beam then $I = \frac{1}{12} b \cdot d^3$.

I_s = moment of inertia of the steel about the neutral axis of the transformed section (in the transformed section the steel is reduced to concrete by multiplying the area of the steel by n).

d' = distance from maximum compression side to the center of area of steel A'.

(2) *Both Tensile and Compressive Stresses.*—(a) If the tension in the concrete is so small as to be permissible the unit stresses may be calculated as in Case (1). The maximum tension in the concrete will be

$$c' = \frac{N}{b \cdot h + A \cdot n + A' \cdot n} - \frac{M'(h - k \cdot d)}{I_c + n \cdot I_s} \tag{160}$$

(*b*) If the tension in the concrete is too large to be carried it will be on the safe side to design the beam so that the compression in the concrete due to the force N, and the bending moment as given by equation (156) is within safe limits, and that the combined stress in the steel is within safe limits, and approximately

$$c = \frac{M'}{(0.19 + 10.5p')b \cdot d^2} + \frac{N}{b \cdot h + n(A + A')} \tag{161}$$

$$f = -\frac{M'}{0.85p \cdot b \cdot d^2} + \frac{N \cdot n}{b \cdot h + n(A + A')} \tag{162}$$

STRESSSES IN COLUMNS.—Concrete columns are seldom made with a length of more than 12 to 15 times the least width, while experiments show that there is little difference in the strength of columns up to ratios of 20 to 25. Columns are reinforced by means of longitudinal rods extending the full length of the column; by means of bands either in the form of hoops or a continuous spiral; or with both longitudinal and hooped reinforcement.

The following notation will be used:

A = total cross-section of a plain concrete column.
A_c = cross-section of the concrete in a reinforced concrete column.
A_s = cross-section of the steel.
c = stress in the concrete.
f = stress in the steel.
$p = A_s/A$.
$n = E_s/E_c$.
P = total strength of a plain concrete column for stress c.
P' = total strength of reinforced concrete column for stress c.

Then

$$P = c \cdot A$$

and

$$P' = c \cdot A_c + f \cdot A_s \tag{163}$$

$$= c(A - p \cdot A) + c \cdot n \cdot p \cdot A \tag{164}$$

$$= c \cdot A[1 + (n - 1)p] \tag{165}$$

The ratio of the strength of the reinforced to the plain concrete column is

$$P'/P = 1 + (n - 1)p \tag{166}$$

Now if $n = 20$ and $p = 1$ per cent, then $P'/P = 1.19$, or an increase of 19 per cent. The ultimate strength of 1-2-4 Portland cement concrete in columns may be taken at 1,600 lbs. per sq. in.

Columns with Hooped Reinforcement.—Experiments show that hooped reinforcement adds considerably to the ultimate strength of

reinforced columns but that very little stress can be developed in the steel for ordinary conditions. Hooped reinforcement is, however, quite effective in preventing sudden failure of columns. Considere gives the following formula for the strength of hooped columns

$$P' = c \cdot A + 2.4\, p \cdot f \cdot A \qquad (167)$$

in which the hooped reinforcement is counted as worth 2.4 times as much as longitudinal reinforcement.

In calculating the strength of reinforced concrete columns the concrete outside the reinforcement is neglected in calculating the strength of the column, this concrete being for protection only.

From an extensive series of experiments Professor A. N. Talbot found that the ultimate strengths of hooped columns were given by the following formulas:

$$\text{for mild steel, } P' = 1{,}600 + 65{,}000p$$

$$\text{for high steel, } P' = 1{,}600 + 100{,}000p$$

where P' = strength per square inch, and p = percentage of steel in terms of the concrete within the hoops. The strength of the plain concrete is taken at 1,600 lbs. per sq. in.

Plain or Deformed Bars.—For ordinary conditions little is to be gained by using deformed bars in the place of plain bars. However, where very high steel is used, or where the dynamic effect of the live load requires consideration, it gives an added degree of safety to use deformed bars. Where a spandrel filling or ballast is used on top of the arch ring, the dynamic effect of the live load on highway and electric railway bridges is a very small percentage of the live load, and may ordinarily be neglected altogether. With ordinary unit stresses in concrete and steel little is to be gained by the use of steel with a high elastic limit.

Methods of Reinforcement.—Many of the arches which have so far been constructed have been built according to some patented system. In the Monier system the reinforcement consists of wire netting, one layer being placed near the extrados and the other near the intrados of the arch. The wires following the arch ring are larger than the transverse wires, the latter being intended to take the temperature and setting stresses.

In the Melan system the steel is in the form of ribs of rolled

I-sections, or built up lattice girders, spaced 2 to 3 feet apart. Mr. Edwin Thacher, M. Am. Soc. C. E., has constructed most of the arches built under this system in this country.

The Kahn system is shown in Fig. 254, and the Luten system is shown in Figs. 255 to 257.

Many arches are now being built in which bars of any satisfactory form are employed in a manner to meet the requirements of the problem. The principal value of reinforcement in a concrete arch is in preventing cracks due to settlement and in making it possible to use higher working stresses in the concrete than would be permissible with plain concrete. On account of the difficulty in exactly locating the line of resistance, it is considered the best practice to reinforce both the intradorsal and extradorsal sides of the arch ring.

REINFORCED CONCRETE BEAM BRIDGES.—For short spans up to 15 to 20 feet beam bridges are commonly made of a single slab of uniform thickness spanning the opening. For longer spans the floor system is made of deeper horizontal girders supporting a relatively thin floor slab. Beam bridges for single spans are designed as simple beams and should be investigated for shearing stresses. The rods should be securely fastened to the end walls. The end walls should be designed as retaining walls to take the thrust of

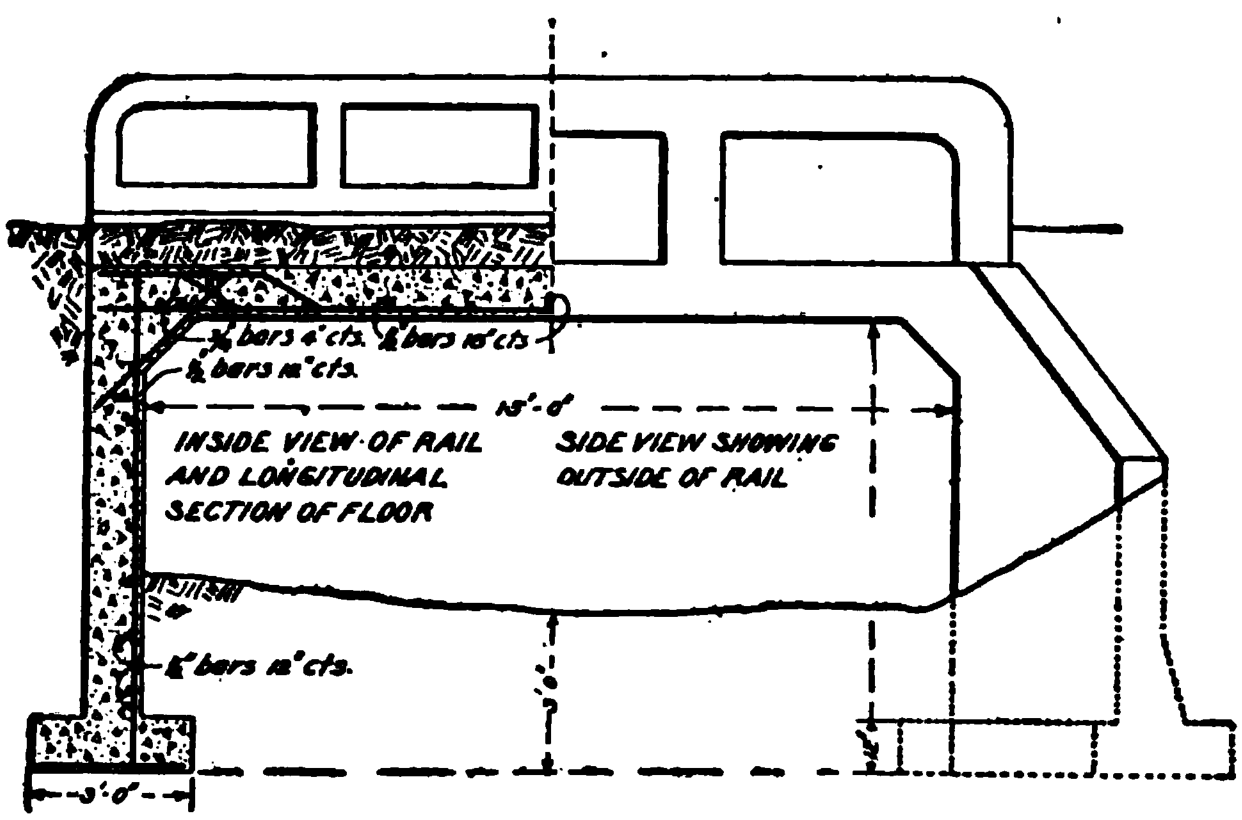


FIG. 249. FIFTEEN-FT. SPAN REINFORCED CONCRETE BEAM HIGHWAY BRIDGE.

the earth fill, for which see the author's book "The Design of Walls, Bins and Grain Elevators." The required live loads for highway and electric railway bridges are given in Appendix I.

Examples of Reinforced Concrete Beam Bridges.—A concrete bridge with 15-ft. span, designed by the Illinois State Highway Commission, A. N. Johnson, highway engineer, is shown in Fig. 249. The concrete was composed of one part Portland cement, 2½ parts sand, and 5 parts broken stone; the rock to be broken so that all would be retained on a ¾ in. screen and all pass a 1½ in. screen. Bars with mechanical bond were used in this bridge.

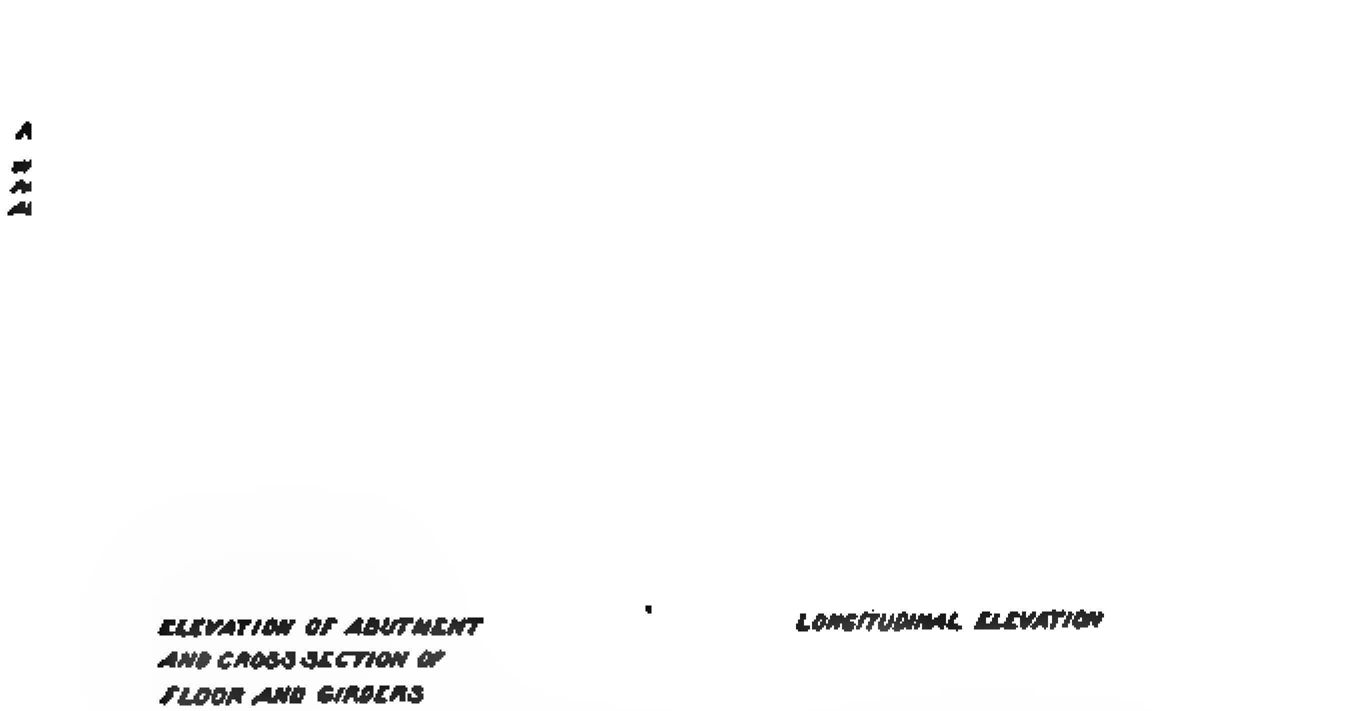


FIG. 250. THIRTY-FT. SPAN REINFORCED CONCRETE BEAM HIGHWAY BRIDGE.

A concrete bridge with a span of 30 ft., as designed by the Illinois Highway Commission, is shown in Fig. 250. A bridge similar to this, built for Tazewell County, Illinois, with 16-ft. clear roadway and 10 ft. from floor to bed of stream, contained 90 cu. yds. of concrete and 8,600 lbs. of steel reinforcement. The contract price for this bridge was $1,125.00.

A Steel Concrete Viaduct.—The steel foot viaduct in Fig. 251 and Fig. 252 was erected at Cedar Rapids, Iowa. The structure is 341 ft. long and has the steel trusses connected by floorbeams and the lower lateral system encased in concrete, and a reinforced concrete floor. There is no provision made for the expansion and contraction of the structure. The bridge is designed for a live load of 40 lbs. per sq. ft. The chords of the trusses are composed of two angles 6″ × 3½″, the webs are single angles 3″ × 2½″, while the posts resting on the piers are two angles 4″ × 3″. The trusses are encased in a sheathing of

⅞″ matched planks, 6 ins. wide. This sheathing is covered with wire netting made of No. 18 wire, 2½″ mesh, kept about ⅜ in. away from the sheathing by lathing. The plaster coat is 1 in. thick and is made of one part Portland cement and two parts sand, and is then given two finishing coats of grout. The reinforced concrete floor is 12 in. thick, reinforced by wire netting and by the lower lateral system. The concrete in the floor is composed of 1–2½–5 Portland cement concrete, with a wearing coat of 1–2 Portland cement, 1 in. thick. This bridge was designed by Mr. J. W. Schaub, M. Am. Soc. C. E., and cost complete about $7,500.00.

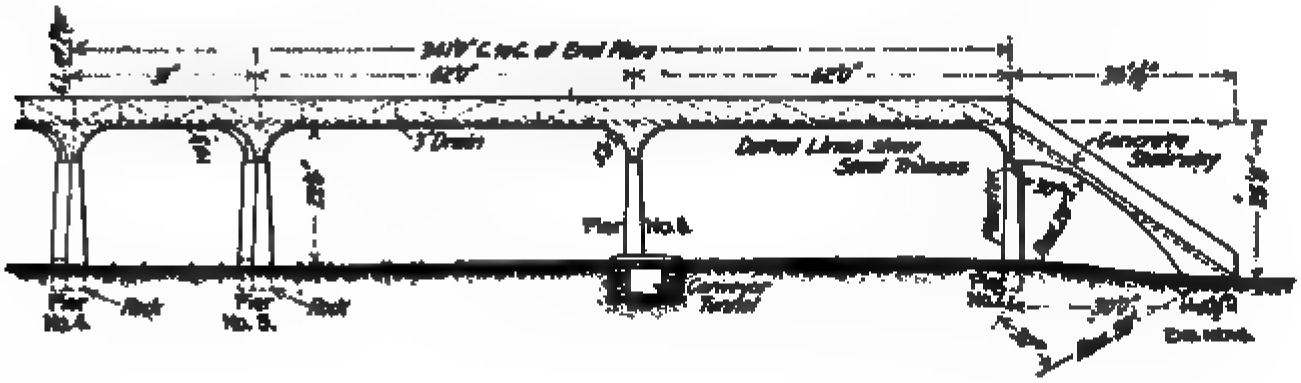


FIG. 251. STEEL CONCRETE FOOT VIADUCT, ELEVATION.

FIG. 252. STEEL CONCRETE FOOT VIADUCT, CROSS-SECTION.

REINFORCED CONCRETE ARCH BRIDGES.—The dimensions of a masonry arch are first assumed, and the arch ring is then investigated for the given loads. Many empirical rules have been given for calculating the thickness of the crown of an arch. The following formula given by Mr. Emile Low, M. Am. Soc. C. E., in

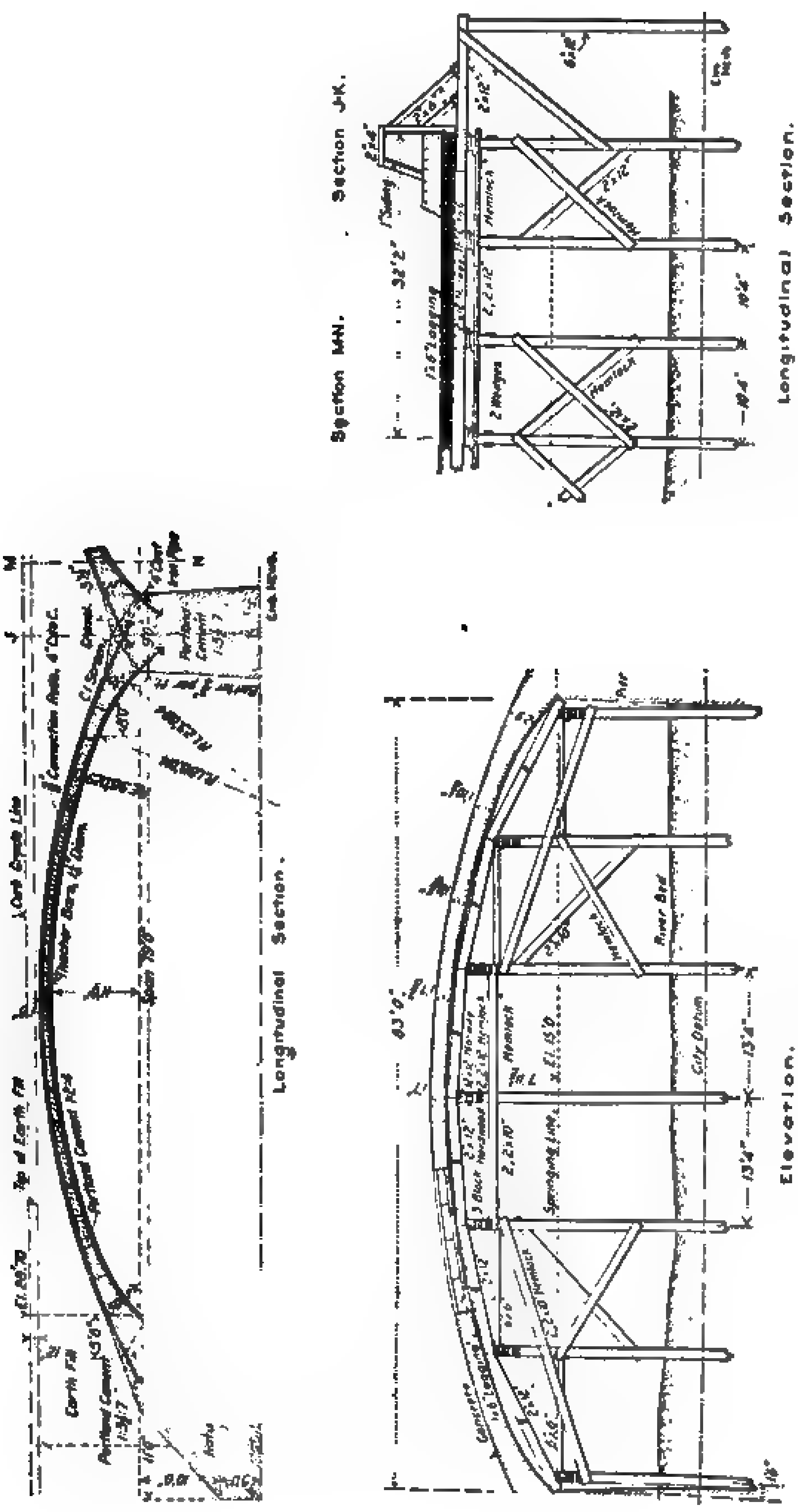


FIG. 253. REINFORCED CONCRETE HIGHWAY BRIDGE, GRAND RAPIDS, MICH.

Eng. News, Nov. 23, 1905, gives quite satisfactory trial values for the thickness of reinforced concrete highway bridges.

$$d = \tfrac{1}{8}\sqrt{10(S-R)+2H}$$

where S = span in ft.; R = radius of arch in ft.; H = surcharge above crown of arch in ft.; d = crown thickness in ft.

In calculating the stresses in the arch ring due to live loads, it is necessary that we know the position of the live load that will produce the maximum stresses. For a uniform live load the maximum stresses ordinarily occur with, from $\frac{3}{8}$ to $\frac{5}{8}$ of the span covered with live load. It is usual to assume that the maximum stresses occur with half of the span covered with the live load.

Reinforced Concrete Arch Bridge.—The reinforced concrete highway bridge at Grand Rapids, Mich., shown in Fig. 253 has 5 arch spans, one 87 ft., two 83 ft. and one 79 ft., and a width of 64 ft. out to out. Details of the 79-ft. arch, the thickness of arch ring for one of the 83-ft. spans, and the arrangement of the falsework are shown in Fig. 253. The concrete was assumed to weigh 150 lbs. per cu. ft., the earth filling 120 lbs. per cu. ft., and the pavement 150 lbs. per cu. ft. The center 20 feet of the bridge was designed for a live load of 250 lbs. per sq. ft., the remainder of the roadway for a live load of 150 lbs. per sq. ft., and the sidewalks for a live load of 100 lbs. per sq. ft. The concrete in the arch ring was 1–2–4 Portland cement concrete, the piers, spandrels and retaining walls were made of 1–3½–7 Portland cement concrete, while the lowest part of the abutments was made of 1–2–4 natural cement concrete.

Allowable Stresses.—The following data were used in the design of the bridge: Modulus of elasticity of concrete, 1,500,000 lbs. per sq. in.; modulus of elasticity of steel, 30,000,000 lbs. per sq. in.

Maximum Compression on Concrete.—Exclusive of temperature stresses, allowable compression was 500 lbs. per sq. in. on 1–2–4 Portland cement concrete. Including stresses due to 40° variation in temperature, the allowable compression was 600 lbs. per sq. in. on 1–2–4 Portland cement concrete. The different parts of the bridge were designed to have a factor of safety of 4 in one month.

Maximum Tension on Concrete.—In arches, exclusive of temperature stresses, allowable tension was 50 lbs. per sq. in. for 1–2–4 Portland cement concrete. In arches, including stresses due to variation of 40° in temperature, allowable tension was 75 lbs. per sq. in. on

1–2–4 Portland cement concrete. In slabs, girders, beams, floors, walls and posts, allowable tension was zero lbs. per sq. in.

Maximum Shear Allowed on Concrete.—The maximum shear on 1–2–4 Portland cement concrete, 75 lbs. per sq. in.

Maximum Stress in Steel. In Arches.—The steel ribs under a stress not exceeding 18,000 lbs. per sq. in. must be capable of taking the entire bending moment of the arch without aid from the concrete, and have flange areas of not less than one one-hundred and fiftieth part of the total area of the arch at the crown. The actual stress in the steel when imbedded in and acting in combination with the concrete, shall not exceed 20 times the allowed stress on the concrete.

In slabs, girders, beams, floors and walls, subject to transverse stress, the steel was assumed to take the entire tensile stress without aid from the concrete and was to have an area sufficient to be equal to the compressive strength of 1–3–6 Portland cement concrete at the age of six months.

In walls and posts subjected to compression only, no allowance was made for the strength of imbedded steel, which was used only as a precaution against cracks due to shrinkage or changes in temperature.

Reinforcement.—Each arch ring was reinforced by 1¼ in. Thacher rods in pairs, consisting of an intradorsal and an extradorsal rod each. The rods are placed 3 in. from the surface of the concrete, and the pairs are spaced 14 in. centers. At the abutment and pier ends the reinforcing rods are fitted with 3 in. washers and nuts as shown, while the rods are made continuous through each arch by means of turnbuckles. The upper and lower rods of each pair are connected by means of ⅝ in. rods with hooked ends, spaced 4 ft. centers.

The concrete was mixed wet and was worked under the bars with rammers. The arch ring was built in transverse sections, and each section was completed in a continuous operation in one day. The crown section was built first, then the two skewbacks, and then the intermediate section.

The bridge was designed and built by the Concrete-Steel Company, New York, N. Y.

Parabolic Reinforced Concrete Arch Bridge.—The reinforced concrete arch bridge across Charley Creek, near Wabash, Indiana, is 240 ft. long, has a width of 32 ft. over all and has two 75-ft. parabolic arch spans with 18-ft. rise, Fig. 254. The arch ring is 18 in. thick at the crown and 3 ft. 4 in. thick at the haunches. It is reinforced with

A

Longitudinal Section.

Section A-B.

Section C-D.

Longitudinal Section.

Cross Section.

FIG. 254. REINFORCED CONCRETE ARCH HIGHWAY BRIDGE ACROSS CHARLEY CREEK, WABASH, IND.

Kahn trussed bars, arranged one above the other in pairs, so that their diagonals interlace across the depth of the arch ring. A number of ¾ in. round rods are inserted transversely in the arch ring to resist temperature. The spandrel walls were designed as vertical cantilever

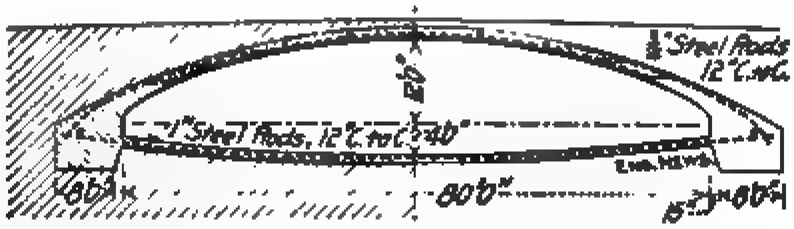


FIG. 255. CLIFTY CREEK REINFORCED CONCRETE ARCH BRIDGE.

slabs. The reinforcement consists of trussed bars set upright and ¾ in. round temperature bars. The arch ring was made of 1–2–4 Portland cement stone concrete, while the spandrel walls and foundations were made of 1–2½–5 Portland cement stone concrete. All concrete was mixed very wet. The bridge was designed and built by the Trussed-Concrete Steel Company, Detroit, Mich.

FIG. 256. SEVENTEENTH STREET BRIDGE, BOULDER, COLORADO. SPAN, 70 FEET; RISE, 10 FT. 6 INS.; ROADWAY, 24 FT.

Luten Arch.—The reinforced concrete highway arch over Clifty Creek, near Greensborough, Indiana, shown in Fig. 255 was designed and built by the National Bridge Company on the Luten system. In this system plain rods are placed near the intrados at the crown and

near the extrados at the haunches, the rods being bent up as shown. The abutments are tied together by horizontal rods imbedded in a concrete pavement. Transverse rods are placed in the arch ring to take the temperature stresses. The arch shown in Fig. 255 has a span of 80 ft., a roadway of 16 ft., contains 265 cu. yds. of concrete, 4,500 lbs. of reinforcement steel in the abutment ties and 4,800 lbs. of steel in the arch and spandrels. The reinforcement was smooth steel rods and cost 2 cts. per lb. in place. The total contract price of the bridge was $2,695.

The 70-ft. span reinforced concrete arch highway bridge at Boulder, Colorado, shown in Figs. 256 and 257 was designed and built by the National Bridge Company on the Luten system. It has a roadway of 24 ft. in the clear and a rise of 10½ ft. The contract price of this bridge was $5,025. For a description of the construction of this bridge, see article by Professor H. C. Ford, in University of Colorado Journal of Engineering, No. 3.

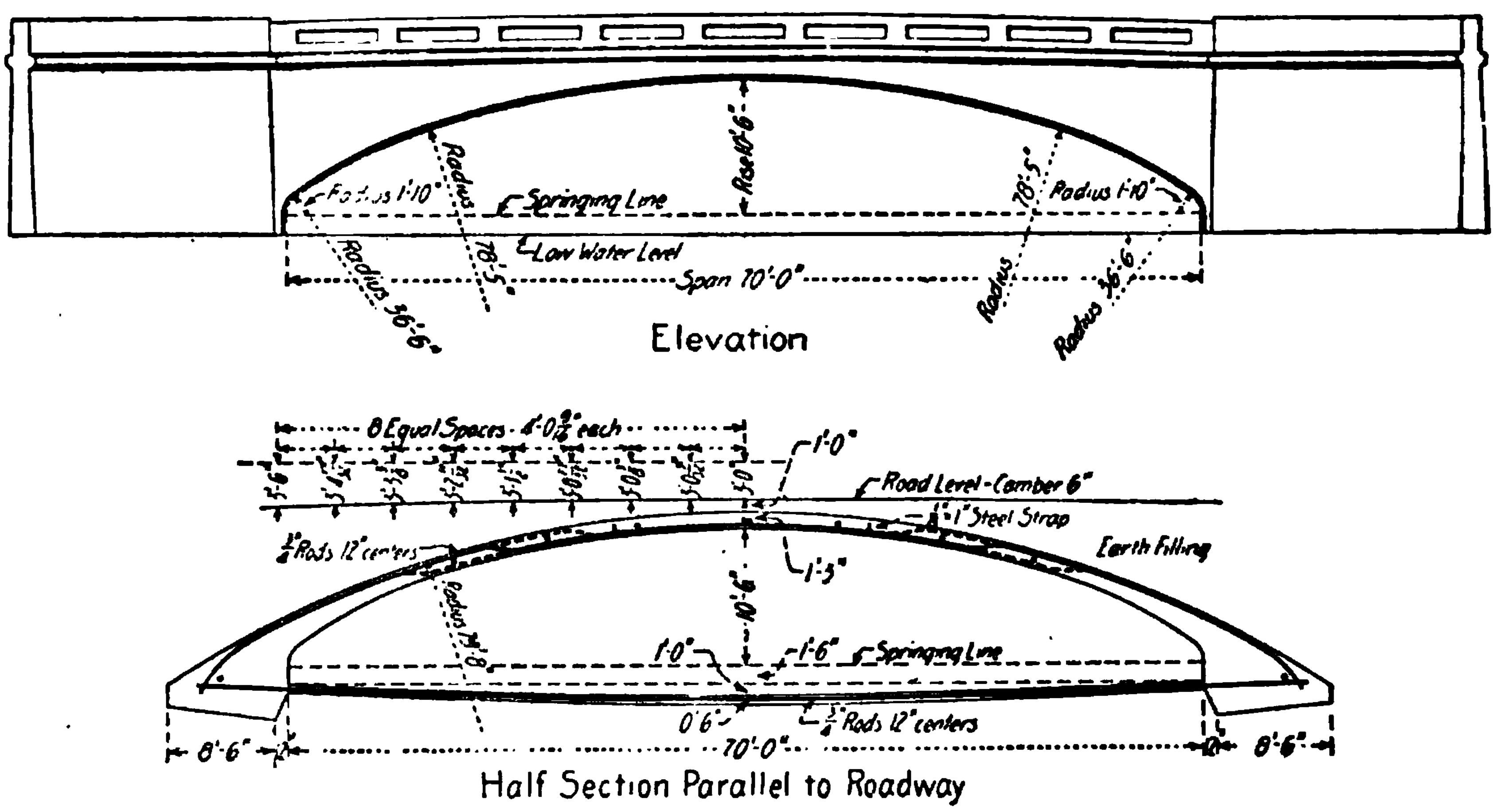


FIG. 257. ELEVATION OF SEVENTEENTH STREET BRIDGE, BOULDER, COLORADO.

The Pike Street bridge, shown in Fig. 258, was designed and erected on the Luten system by the National Bridge Company. The span is 30 ft.; rise, 6 ft.; height from pavement to crown of road, 7 ft. 6 in.; roadway, 60 ft.; thickness at the crown, 12 in.; pavement rods, ¾ in., spaced 3 ft. apart in 6 in. of concrete; arch rods, ¾ in., spaced 3 ft. apart. Contract price was $2,280.00; cost to the contractor, $2,073.00.

The actual cost of the Boulder, Colo., bridge was as follows:

Item.	Cost.	Proportion of Total Cost in Per Cent.
Removing old bridge	$ 144.00	2.86
Excavation	200.00	3.98
Lumber	480.00	9.56
Steel for reinforcement, 9,000 lbs. @ $0.041	370.00	7.37
Cement, 390 bbls. @ $2.525	1,015.00	20.21
Gravel, 470 yds. @ $0.67	315.00	6.27
Concrete labor, including placing of steel	360.00	7.16
Welding steel	40.00	0.80
Erection of forms	310.00	6.17
Removing forms and back filling	286.28	5.70
Sidewalks	135.00	2.68
Freight on tools, Indianapolis to Boulder and return	227.16	4.52
Supervision	415.00	8.26
Miscellaneous	159.06	3.16
Depreciation (estimated)	100.00	1.99
Contractor's profit	468.50	9.31
Contract price	$5,025.00	100.00

FIG. 258. PIKE STREET REINFORCED CONCRETE BRIDGE, PONTIAC, MICH. SPAN, 30 FT.; RISE, 6 FT.

For cuts and information the author is under obligations to Mr. Daniel B. Luten, Consulting Engineer, President National Bridge Co.

THE DESIGN OF CULVERTS.—The required size of a culvert for any particular opening is difficult to determine by calculation. The required waterway will depend upon the size of the drainage area, the

slope of the surface of the area, the character of the soil, and upon the design of the culvert. A larger opening will be required for a culvert in a low embankment than in a high, well-compacted embankment.

Many formulas have been proposed for determining the waterway of culverts. The best formula known to the author is that proposed by Professor A. N. Talbot. It is

$$A = c\sqrt[4]{M^3}$$

where A = area of the required opening in sq. ft.;
M = area of drainage basin in acres;
c = a coefficient varying with the slope of the ground, slope of the drainage area, character of the soil and character of vegetation.

TABLE LVIII.

APPROXIMATE AREA AND SIZE OF WATERWAY (SANTA FE RAILWAY).

Area Drained in Square Miles.	Area of Waterway in Square Feet.	Box and Arch Culverts. Dimensions in Feet.	Area Drained in Square Miles.	Area of Waterway in Square Feet.	Arch Culverts. Dimensions in Feet.
0.01	2.0	2′ 0″ × 1′ 0″ Box	1.5	150	14′ 0″ × 7′ 0″ Arch
0.02	4.0	2 0 × 2 0 Box	2.0	200	18 0 × 7 0 Arch
0.03	6.0	3 0 × 2 0 Box	2.5	250	20 0 × 8 0 Arch
0.04	7.5	3 0 × 2 6 Box	3.0	300	20 0 × 9 6 Arch
0.05	9.0	3 0 × 3 0 Box	3.5	350	24 0 × 8 6 Arch
0.06	10.5	3 6 × 3 0 Box	4.0	388	28 0 × 7 0 Arch
0.07	12.0	4 0 × 3 0 Box	5.0	455	28 0 × 9 6 Arch
0.08	13.5	2 6 × 3 0 Box	6.0	509	32 0 × 7 6 Arch
0.09	15.0	2 6 × 3 0 Box	7.0	556	32 0 × 9 0 Arch
0.10	16.0	3 0 × 3 0 Box	8.0	601	32 0 × 11 0 Arch
0.15	25.0	3 0 × 4 0 Box	9.0	641	32 0 × 12 0 Arch
0.20	32.0	6 0 × 4 0 Arch	10.0	679	32 0 × 13 0 Arch
0.30	44	6 0 × 5 6 Arch	15.0	835	
0.40	56	8 0 × 5 0 Arch	20.0	970	
0.50	66	8 0 × 6 0 Arch	30.0	1,180	
0.60	74	10 0 × 4 6 Arch	40.0	1,350	
0.70	81	10 0 × 5 6 Arch	50.0	1,510	
0.80	88	10 0 × 6 6 Arch	100.0	2,120	
0.90	94	10 0 × 6 6 Arch			
1.00	100	12 0 × 5 0 Arch			

Professor Talbot gives the following values of c: $c = \frac{2}{3}$ to 1 for steep and rocky ground; $c = \frac{1}{3}$ for rolling agricultural country, subject

to floods at times of melting snow, and with the length of valley 3 to 4 times its width; $c = \frac{1}{4}$ to $\frac{1}{8}$ for districts not affected by accumulated snow and where the length of the valley is several times its width.

Quite a number of railroad companies have prepared tables for the use of their engineers in designing the waterway of culverts and bridges. The standards for waterways of culverts on the Santa Fe system are given in Table LVIII.

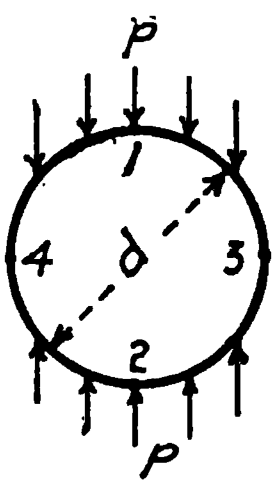


FIG. 259.

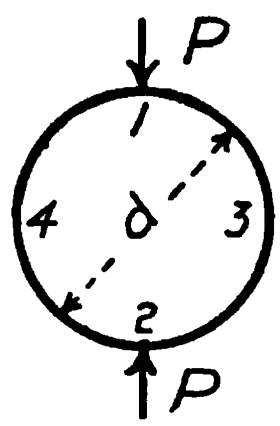


FIG. 260.

It should be remembered that formulas and data such as given in Table LVIII are to be used only as aid to the judgment where definite information on the amount of water passing the point cannot be obtained. Before determining on the size of a given culvert a careful investigation should be made at the site to determine high water marks and other data to be used in this connection.

STRESSES IN CULVERTS.—The loads coming on the structure cannot be accurately calculated and an exact analysis is therefore impossible. Arch culverts are designed in the same manner as arch bridges. The design of pipe and box culverts will be considered briefly, the details of the analysis being omitted. The following analysis is essentially the same as given in Chapter X, "Principles of Reinforced Concrete" by Turneaure and Maurer. For a discussion of an extensive series of tests of culvert pipe by Professor A. N. Talbot, see University of Illinois Engineering Experiment Station Bulletin, No. 22.

Pipe Culverts.—There will be two cases: (1) a uniform load, and (2) a concentrated load.

(1) *Uniform Load.*—It is assumed that the pressure acts in parallel lines, Fig. 259.

Let d = diameter of the pipe in inches;

p = the load on the pipe in lbs. per sq. in.;

M = bending moment in the pipe in the length of one inch in in.-lbs.

Then

$$M_1 = M_2 = \tfrac{1}{16}\, p \cdot d^2 \qquad (168)$$

$$M_3 = M_4 = -\tfrac{1}{16}\, p \cdot d^2 \qquad (169)$$

If the lateral pressure be q per unit of area, then

$$M_1' = M_2' = -\tfrac{1}{16}\, q \cdot d^2 \qquad (170)$$

$$M_3' = M_4' = \tfrac{1}{16}\, q \cdot d^2 \qquad (171)$$

If $p = q$ the bending moments are zero. The relation between the vertical and lateral pressure is given by the equation

$$q = p \cdot (1 - \sin \phi)/(1 + \sin \phi)$$

where ϕ is the angle of internal friction of the filling. For $\phi = 30°$, $q = \tfrac{1}{3} p$. For this case the bending moments will be

$$M_1 = M_2 = \tfrac{1}{16}\, p \cdot d^2 - \tfrac{1}{48}\, p \cdot d^2 = \tfrac{1}{24}\, p \cdot d^2 \qquad (172)$$

$$M_3 = M_4 = -\tfrac{1}{16}\, p \cdot d^2 + \tfrac{1}{48}\, p \cdot d^2 = -\tfrac{1}{24}\, p \cdot d^2 \qquad (173)$$

(2) *Concentrated Loads.*—In this case, Fig. 260,

$$M_1 = M_2 = 0.16\, P \cdot d \qquad (174)$$

$$M_3 = M_4 = -0.09\, P \cdot d \qquad (175)$$

Box Culverts. (1) *Uniform Loads.*—Let

w = width of culvert;
h = height of culvert;
I_1 = moment of inertia of top or bottom slab;
I_2 = moment of inertia of each side slab;
p = vertical load, assumed as the same on the top and bottom, (a) Fig. 261.

Then

$$M_1 = M_2 = \tfrac{1}{8} p \cdot w^2 \cdot (w/(3I_1) + h/I_2)/(w/I_1 + h/I_2) \qquad (176)$$

$$M_3 = M_4 = M_1 - \tfrac{1}{8} p \cdot w^2 \qquad (177)$$

The moments at a and b are equal to M_3.

For a square culvert with uniform section

$$M_1 = \tfrac{1}{12}\, p\cdot w^2 \tag{178}$$

$$M_3 = -\tfrac{1}{24}\, p\cdot w^2 \tag{179}$$

For equal lateral and vertical loads on a square culvert

$$M_1 = M_3 = +\tfrac{1}{24}\, p\cdot w^2, \text{ and } M_a = -\tfrac{1}{12}\, p\cdot w^2 \tag{180}$$

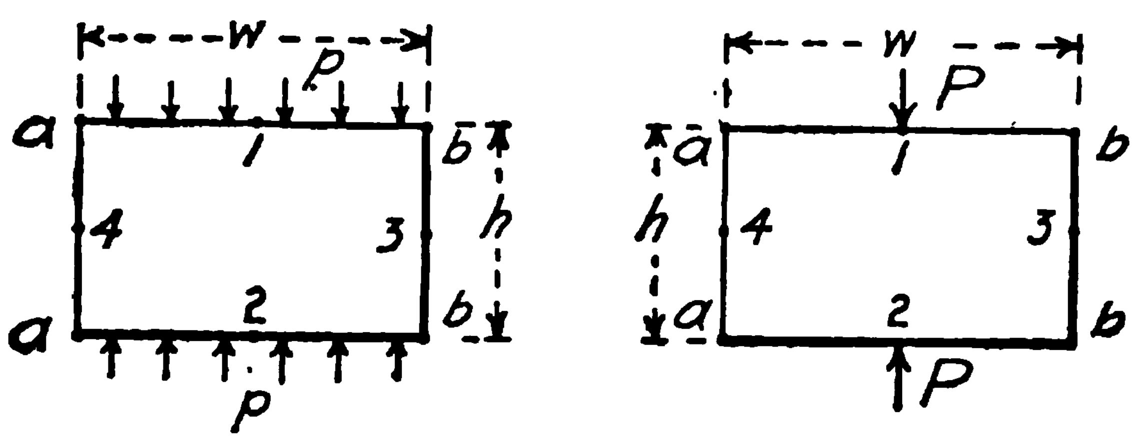


FIG. 261.

(2) *Concentrated Loads.*—For vertical loads applied at the center of the top and bottom, (b) Fig. 261.

$$M_1 = M_2 = \tfrac{1}{4}P\cdot w\cdot (w/(2I_1) + h/I_2)/(w/I_1 + h/I_2) \tag{181}$$

$$M_3 = M_4 = M_1 - \tfrac{1}{4}P\cdot w \tag{182}$$

To provide for the negative moments at the corners proper reinforcement must be used.

TYPES OF CULVERTS.—Culverts are made of timber, vitrified clay pipe, cast iron pipe, riveted steel pipe, brick, stone, concrete and reinforced concrete.

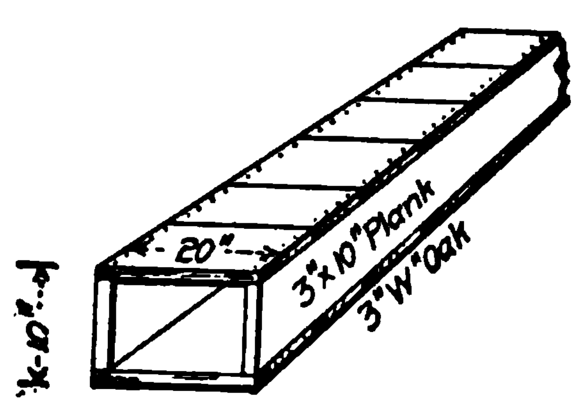


FIG. 262. TIMBER BOX CULVERT.

Timber.—For temporary culverts the timber box culvert, as shown in Fig. 262, or the timber culvert, as shown in Fig. 263, may be used. The bottom of the culvert in Fig. 263 should be paved to prevent scour.

Unless care is used to carefully tamp the filling, the water will flow along the sides of both of the culverts shown. Timber culverts are very unsatisfactory and in the long run are very expensive.

FIG. 263. TIMBER CULVERT.

Pipe Culverts.—Vitrified clay, cast iron, steel plate and concrete pipes are used for culverts. Pipe culverts should be laid on a firm foundation to a careful grade. The center should be raised so that there will be no hollows in the pipe. Head walls, preferably of masonry or concrete should extend high enough to carry the fill, and should be carried down far enough to prevent the water from following along the outside of the pipe. The pipe should preferably be laid in concrete.

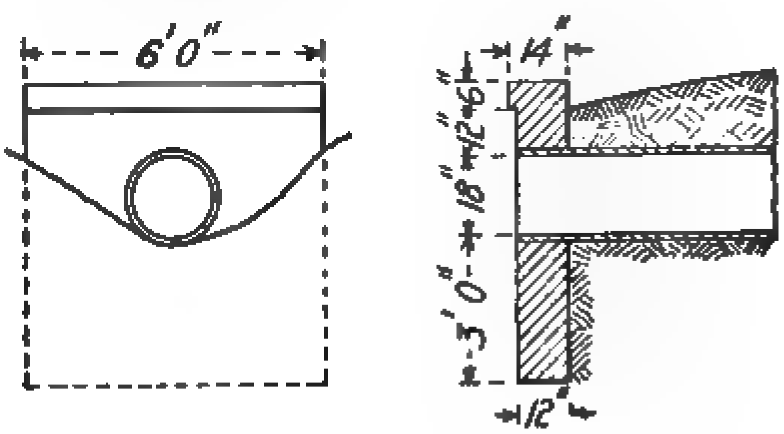


FIG. 264. END WALLS FOR PIPE CULVERTS.

Vitrified Clay Pipe Culverts.—Vitrified clay pipe is made in single and double strength or culvert pipe. The double strength pipe should preferably be used for culverts. The pipe should be laid in a trench

rounded off to fit the pipe with the bells up stream. The joints should be calked with 1–3 Portland cement mortar. The earth should be well tamped around and over the pipe, and in no case should the wheels of wagons be permitted to come nearer to the top than the diameter of the pipe. Both ends of the pipe should be protected by masonry or concrete head walls as shown in Fig. 264.

The common sizes, weights and list prices of vitrified clay pipe are given in Table LIX. The discounts from this list will vary with the location and the weight of the pipe. In the middle west the discounts from the list, delivered in car load lots, are approximately 75 per cent for single strength and 50 per cent for culvert pipe.

TABLE LIX.

SIZE, WEIGHT AND LIST PRICE OF VITRIFIED CLAY PIPE.

Inside Diameter, Ins.	Area, Sq. Ins.	Single Strength.		Double Strength.	
		Weight per ft.	List Price per ft.	Weight per ft.	List Price per ft.
10	78	32	$0.60	50	$0.75
12	113	43	.75	70	1.00
15	177	65	1.00	100	1.50
18	254	86	1.50	125	1.75
20	314	100	1.75	180	2.50
24	452	131	2.50	240	3.25
27	573	215	3.25	300	4.00
30	707	270	4.00	340	5.00
33	855	320	5.00	390	6.00
36	1,018	365	6.00		

Cast Iron Pipe Culverts.—Cast iron pipe for use in culverts can be obtained in 12 ft. lengths, in 3 ft. lengths, or in sectional form, the sections being bolted together in place. Cast iron pipe can be laid nearer the surface than vitrified clay pipe and is not damaged by freezing water. Cast iron pipe should be laid in the same manner as clay pipe and should have substantial end walls. Cast iron pipe is made with different weights per foot, the lighter weights being ordinarily used for culverts. The weight per foot for cast iron pipe is given in Table LX.

The price of cast iron pipe changes with the market conditions, but varies from 1.25 to 1.50 cts. per lb. at the factory, and from 2.00 to 3.00 cts. per lb. delivered.

Steel Plate Pipe Culverts.—Culverts are made of steel plates riveted in a circular or semicircular form. Plate pipe culverts should be laid

TABLE LX.

SIZES, THICKNESS AND WEIGHTS OF CAST IRON PIPE.

Inside Diameter, Inches.	Standard 12 Foot Sections.				Standard 3 Foot Sections.	
	Light.		Medium.			
	Thickness, Inches.	Weight, per Foot. Pounds.	Thickness, Inches.	Weight per Foot. Pounds.	Thickness, Inches.	Weight per Foot. Pounds.
12	½	70	[illegible]	75	⅜	60
14	[illegible]	90	[illegible]	100	7/16	70
16	[illegible]	100	[illegible]	125	½	90
18	[illegible]	135	[illegible]	167	9/16	105
20	[illegible]	150	[illegible]	200	9/16	120
24	[illegible]	200	1	250	⅝	160
30	[illegible]	250	1 1/16	334	¾	220
36	⅞	334	1⅛	450	¾	300

with care and should have masonry head walls. The plates should be not less than 3/16″ thick for small sizes, and up to ⅜″ for culverts 4 feet in diameter and over. The fill should extend above the top of the pipe a distance equal to at least one-half of the diameter of the pipe, and should be well tamped around the sides. Corrugated galvanized culverts are now on the market. The galvanized sheets in these culverts are so thin that the metal will soon corrode. In estimating the cost of steel pipe culverts add 10 to 15 per cent to the weight of the

FIG. 265. REINFORCED CONCRETE CULVERT PIPE, C. B. & Q. R. R.

plates to cover the laps and the rivets. The shop costs of steel plate culvert pipe are given in Chapter XIX. The freight rates on culvert pipe are liable to be high, due to the fact that it is difficult to get sufficient weight on a car to give a minimum car load unless the pipes are of different sizes and can be nested.

Reinforced Concrete Pipe Culverts.—Details of a reinforced concrete pipe for culverts, as designed by C. H. Cartlidge for the C. B. & Q. R. R., are shown in Fig. 265, while the forms for molding a similar

TABLE LXI.

RELATIVE COSTS OF CAST IRON AND REINFORCED CONCRETE PIPE.

Diameter, Inches.	Cast Iron Pipe.			Reinforced Concrete Pipe.		
	Thickness, Inches.	Weight Per Foot Lbs.	Cost Per Foot.	Thickness, Inches.	Weight Per Foot Lbs.	Cost Per Foot.
12	[illegible]	75	$ 2.44	2	88	$0.16
18	[illegible]	167	5.43	3	222	0.36
24	1	250	8.13	$4\frac{1}{2}$	420	0.68
36	$1\frac{1}{4}$	450	14.63	$4\frac{3}{4}$	676	1.10
48	$1\frac{7}{16}$	725	23.50	6	1,131	1.83

pipe are shown in Fig. 266. The relative costs of cast iron and reinforced concrete pipe, as given by Mr. O. P. Chamberlain in Eng. News, Dec. 20, 1906, are shown in Table LXI.

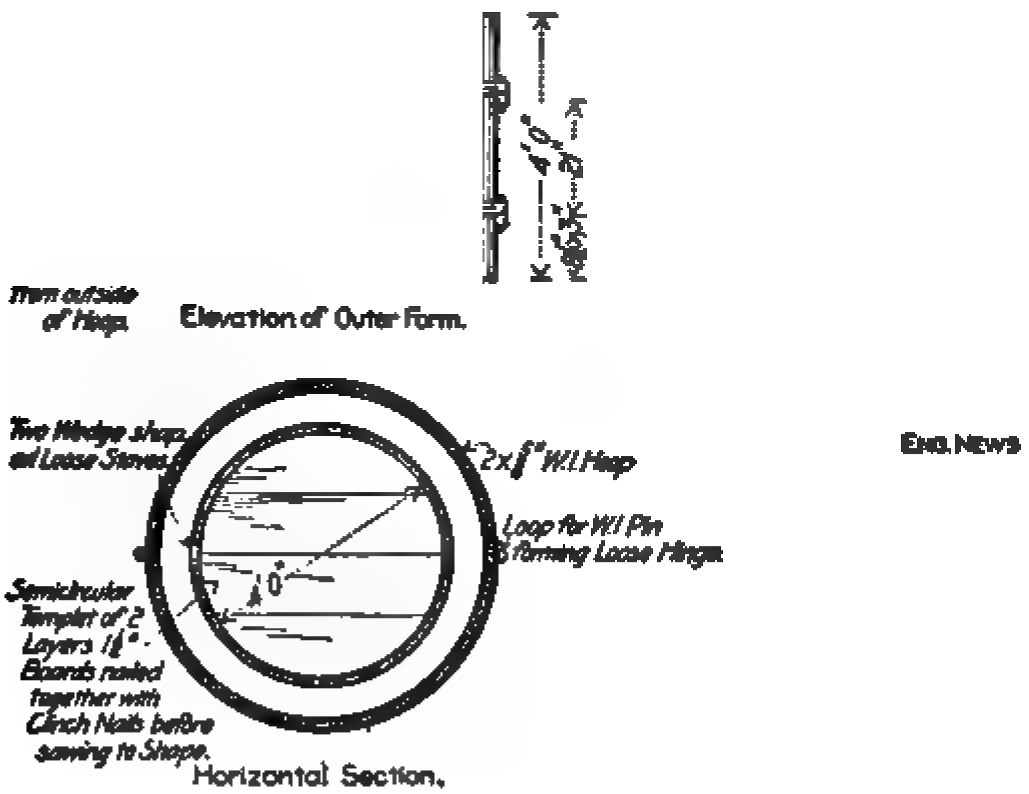


FIG. 266. FORMS FOR MOLDING REINFORCED CONCRETE CULVERT PIPE.

The costs for reinforced concrete pipe are low and should probably be increased 50 to 100 per cent for single culverts in addition to the cost of making the forms.

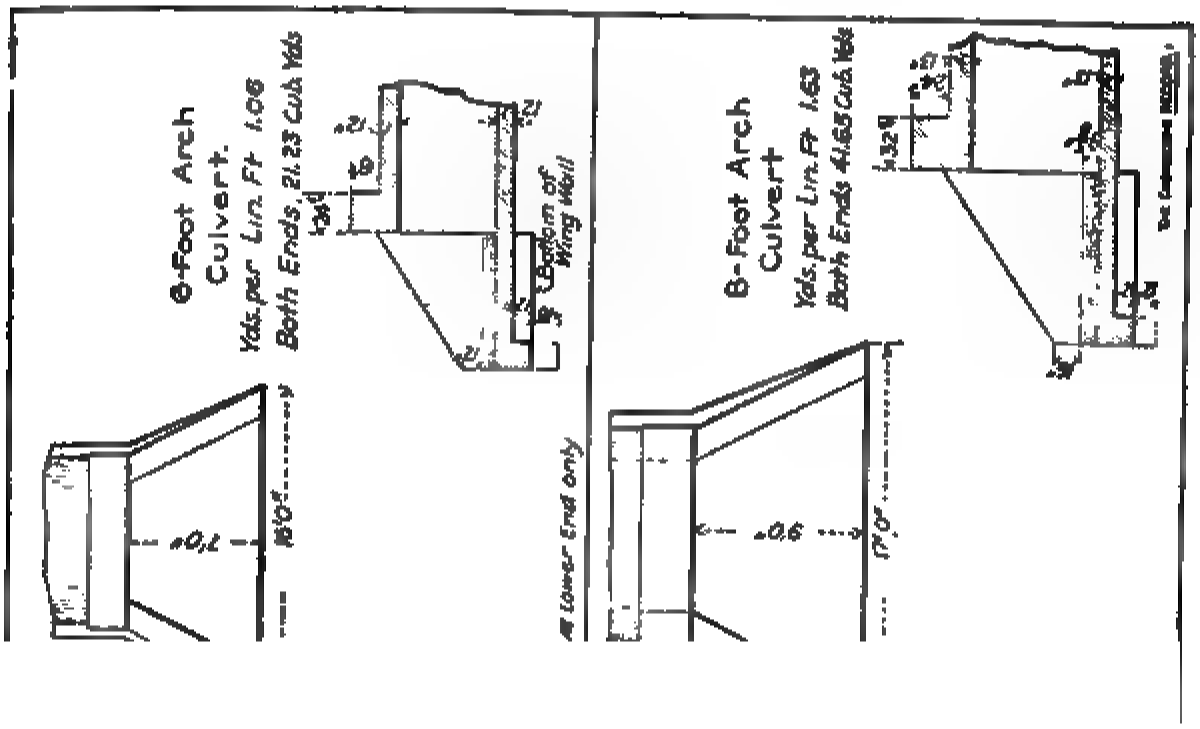


FIG. 267. STANDARD MASONRY CULVERTS, MISSOURI PACIFIC RAILWAY.

Plain Concrete Culverts.—The Missouri Pacific R. R. standard culverts are shown in Fig. 267. These culverts are built of plain concrete and contain quantities as shown. It will be seen that straight walls are used on the small culverts and wing walls on the large culverts.

Plans for an 8-ft. plain concrete culvert, as designed by Edwin Thacher for the highways in Porto Rico, are shown in Fig. 268.

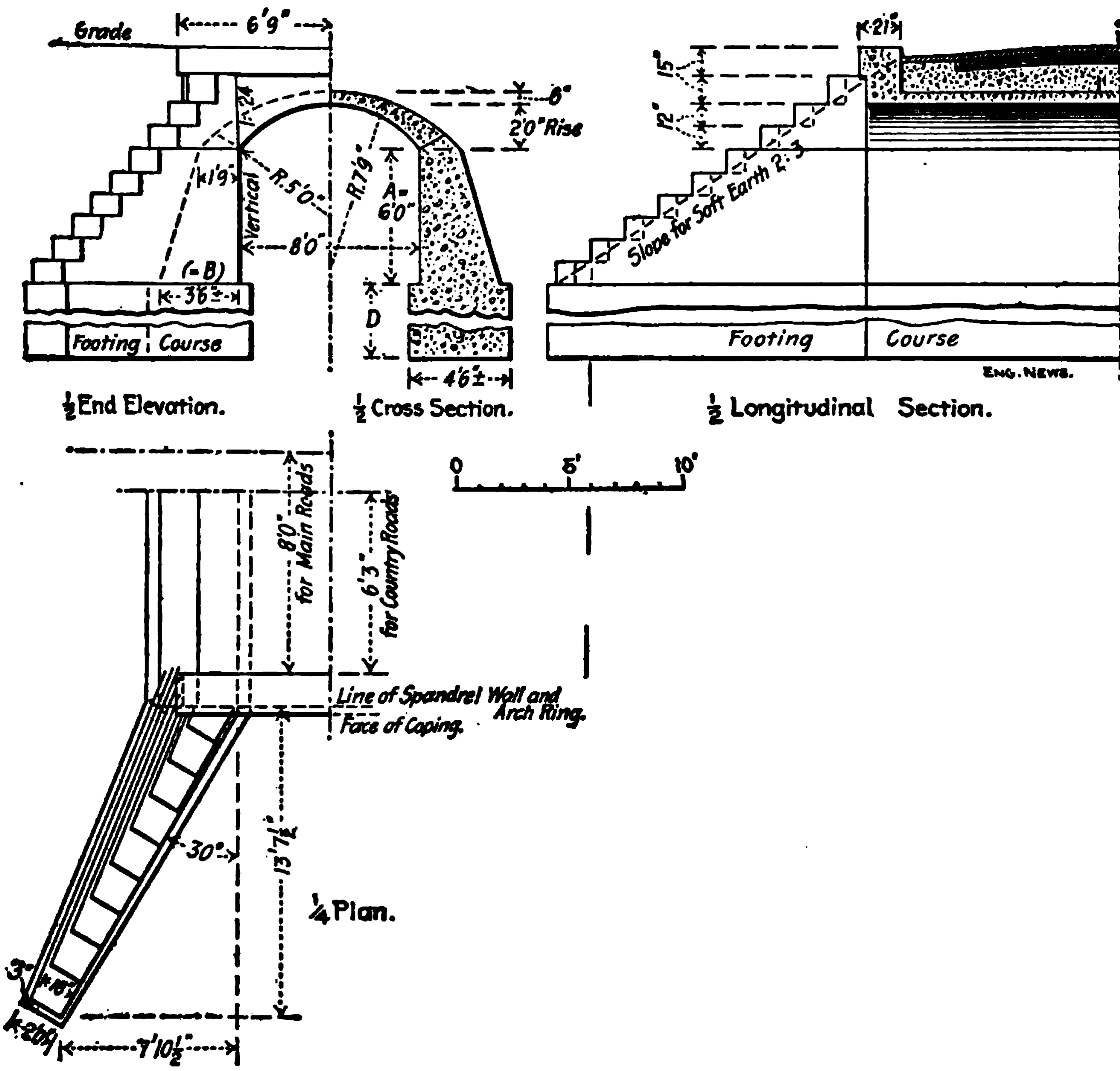


FIG. 268. PLANS OF EIGHT-FOOT PLAIN CONCRETE HIGHWAY CULVERT, PORTO RICO.

Relative Costs of Small Culverts.—The relative costs of small culverts, as calculated by A. R. Hurst, highway engineer, Wisconsin Geological and Natural History Survey, are given in Table LXII.

Reinforced Concrete Culverts.—The plans of a reinforced concrete box culvert 4′ 0″ × 4′ 0″ are given in Fig. 269; of a reinforced

TABLE LXII.

RELATIVE COST OF SMALL CULVERTS OF APPROXIMATELY THE SAME WATERWAY, STANDARD SIZES.*

KIND.	SIZE.	AREA, SQ. IN. OF WATERWAY.	COST PER LINEAL FOOT LAID.	COST PER SQ. FT. OF WATERWAY.	MOST ECONOMICAL LENGTH.	COST OF CULVERT.	COST OF END WALLS.	COST COMPLETE.	COST TO KEEP UP 100 YEARS.
3 inch hemlock wood box...	15 in. sq.	225	$ .70	$ .44	24	$16.80	....	$16.80	$252.00
Concrete box..................	15 in. sq.	225	1.10	.70	20	22.00	$18.00	40.00	40.00
Concrete arch	16 in.	228	1.25	.80	20	25.00	18.00	43.00	43.00
Circular concrete pipe	18 in.	254	.85	.48	20	17.00	18.00	35.00	35.00
Cast iron, 12 feet lengths....	18 in.	254	3.10	1.76	24	74.40	18.00	92.40	166.80
Cast iron, 3 feet lengths......	18 in.	254	3.00	1.70	21	63.00	18.00	81.00	144.00
Single strength vitrified clay.	18 in.	254	.90	.51	30	27.00	14.00	41.00	41.00
Double strength vitrified clay	18 in.	254	1.00	.57	28	28.00	14.00	42.00	42.00
Corrugated steel	18 in.	254	1.40	.80	26	36.40	14.00	50.40	196.00

concrete arch culvert 8′ 0″ × 8′ 0″ are given in Fig. 270; and of a reinforced concrete arch culvert of 20′ 0″ span are given in Fig. 271.

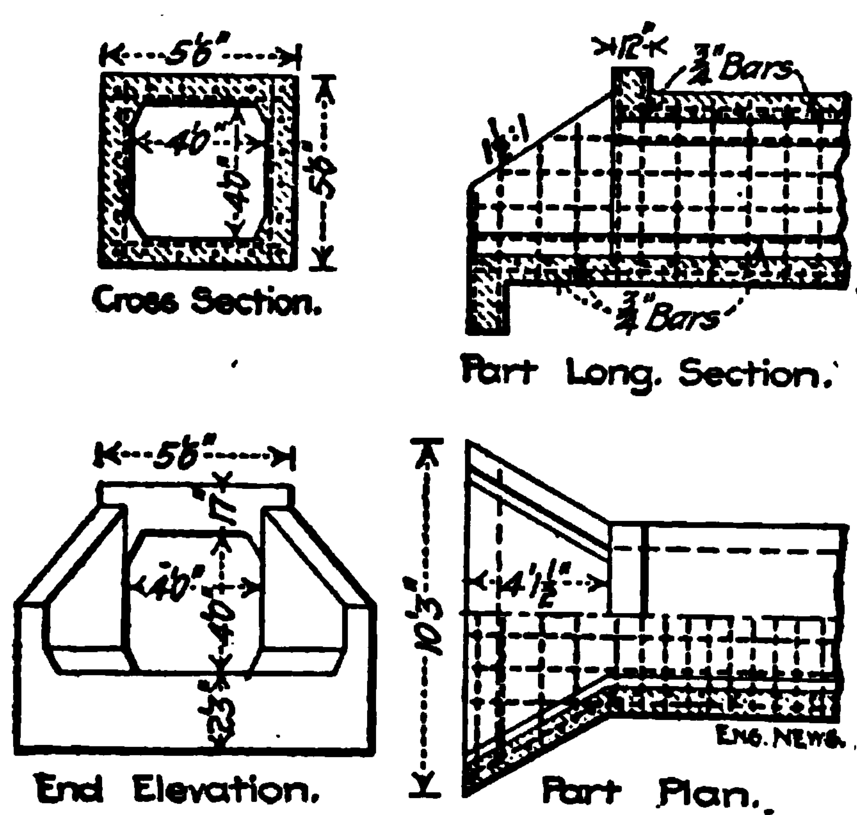


FIG. 269. REINFORCED CONCRETE BOX CULVERT, GREAT NORTHERN RAILWAY.

These culverts were designed by Mr. C. F. Graff for the Great Northern Railway, and are strong enough to carry from 20 to 40 ft. of railroad embankment. The quantities for one pair of wing walls and one foot

* A. R. Hurst, highway engineer, Wisconsin Geological and Natural History Survey.

of barrel of culvert are given in Table LXIII. These culverts are made heavier than is necessary for ordinary highway culverts.

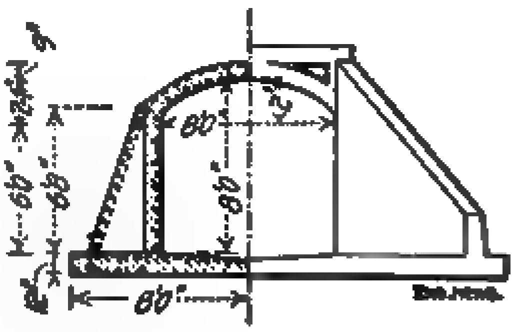


FIG. 270. REINFORCED CONCRETE ARCH CULVERT, GREAT NORTHERN RAILWAY

Cost of Reinforced Concrete Culverts.—The cost of reinforced concrete culverts should be estimated in the same manner as reinforced

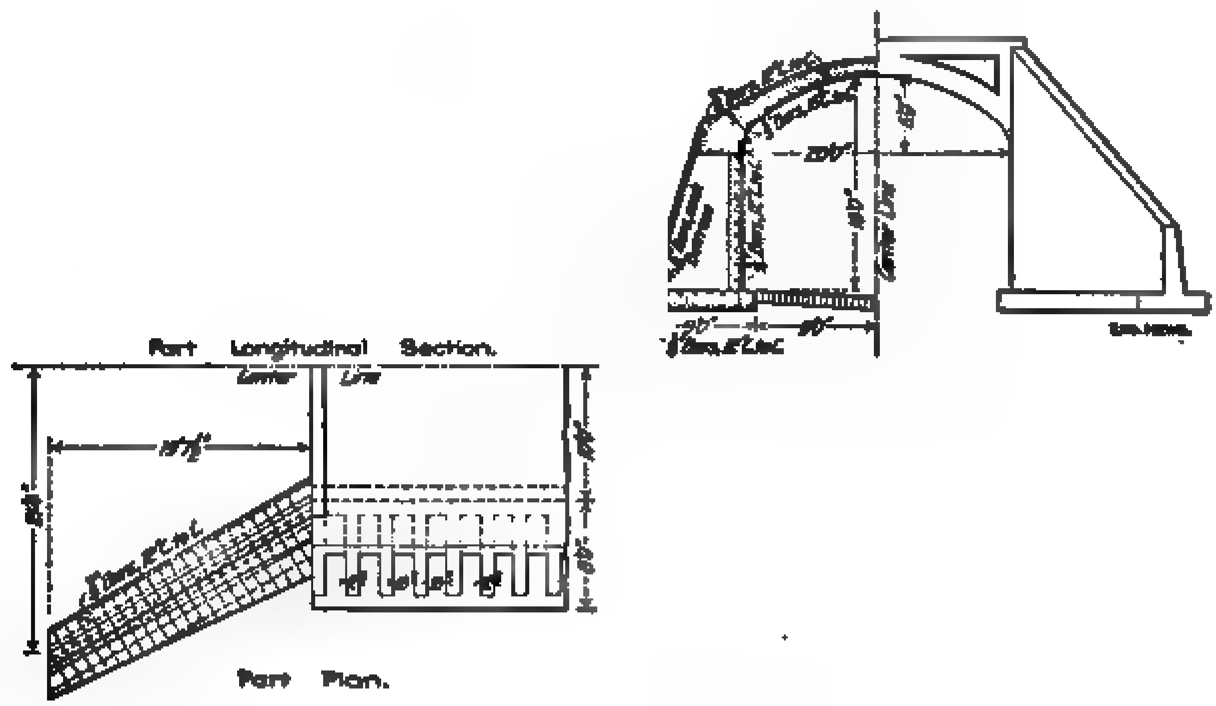


FIG. 271. REINFORCED CONCRETE ARCH CULVERT, GREAT NORTHERN RAILWAY.

concrete arches. For details for estimating the cost of forms, etc., see Chapter XIX.

TABLE LXIII.

QUANTITIES IN REINFORCED CONCRETE CULVERTS; FIGS. 269 TO 271.

Size.	Barrel per Lineal Foot.		One Pair of Wing Walls.	
	Concrete, cubic yards.	Steel, Pounds.	Concrete, cubic yards.	Steel, Pounds.
2′ 0″ circular	0.10	5		
3′ 0″ "	0.23	10		
4′ 0″ "	0.30	12		
4′ × 4′	0.54	61	2.38	141
4′ × 6′	0.72	73	2.50	128
6′ × 6′	0.86	116	5.16	397
8′ × 8′	1.37	158	11.22	793
12′ × 12′	2.78	237	37.25	1,850
17′ × 16′	3.70	287	51.80	2,579
16′ × 20′	5.00	300	45.60	2,143

CHAPTER XVIII.

THE DESIGN OF TIMBER AND COMBINATION BRIDGES.

Introduction.—Timber was formerly quite generally used in the construction of highway bridges, and is still used for temporary structures in locations where timber is cheap and iron and steel are relatively expensive. The subject will be taken up under the following heads: (1) Timber trestles; (2) timber bridges, and (3) combination bridges.

TIMBER TRESTLES.—The following definitions have been adopted by the American Railway Engineering and Maintenance of Way Associtaion:

Wooden Trestle.—A wooden structure composed of upright members supporting simple horizontal members or beams, the whole forming a support for loads applied to the horizontal members.

Frame Trestle.—A structure in which the upright members or supports are framed timbers.

Pile Trestle.—A structure in which the upright members or supports are piles.

Bent.—The group of members forming a single vertical support of a trestle; designed as pile bent where the principal members are piles, and as framed bent where of framed timbers.

Post.—The vertical members of the bent of a framed trestle.

Pile.—Timber driven in the ground, and intended generally to support a structure.

Batter.—A deviation from the vertical in upright members of a bent.

Cap.—A horizontal member upon the top of piles or posts, connecting them in the form of a bent.

Sill.—A lower horizontal member of a framed bent.

Sub-sill.—Timber bedded in the ground to support framed bents.

Intermediate Sill.—A horizontal member in the plane of the bent between the cap and sill to which posts are framed.

Sway Brace.—A member bolted or spiked to the bent and extending diagonally across its face.

Longitudinal Strut or Girt.—A stiff member running horizontally, or nearly so, from bent to bent.

Longitudinal X Brace.—A member extending diagonally from bent to bent in a vertical or battered plane.

Sash Brace.—A horizontal member secured to the posts or piles of a bent.

Stringer.—A longitudinal member extending from bent to bent and supporting the ties.

Jack Stringer.—A single line of stringers placed outside of the main stringers.

Tie.—A transverse timber resting on the stringers and supporting the rails.

Guard Rail.—A longitudinal member, either iron or wood, secured on top of ties.

Packing Block.—A small member, usually wood, used to secure the parts of a composite member in their proper relative positions.

Packing Spool or Separator.—A small casting used in connection with packing bolts to secure the several parts of a composite member in their proper relative positions.

Drift Bolt.—A piece of round or square iron of specified length, with or without head or point, driven as a spike.

Dowel.—An iron or wood pin, extending into, but not through, two members of the structure to connect them.

Shim.—A small piece of wood or metal placed between two members of a structure to bring them to a desired relative position.

Fish-Plate.—A short piece lapping a joint, secured to the side of several members, which are butt-jointed.

Bulkhead.—Timber placed against the side of an end bent for the purpose of retaining the embankment.

Defects of Structural Timber.—The standard defects included in the following list are most such as may be termed natural defects, as distinguished from defects of manufacture. The latter have usually been omitted, because the defects of manufacture are of very minor significance in the grading of structural timber:

Sound Knot.—A sound knot is one which is solid across its face and is as hard as the wood surrounding it. It may be either red or black, and is so fixed by growth or position that it will retain its place in the piece.

Loose Knot.—A loose knot is one not firmly held in place by growth or position.

Pith Knot.—A pith knot is a sound knot with a pith hole not more than $\frac{1}{4}$ in. in diameter in the center.

Encased Knot.—An encased knot is one which is surrounded wholly or in part by bark or pitch. Where the encasement is less than $\frac{1}{8}$ of an inch in width on each side, nor exceeding one-half the circumference of the knot, it shall be considered a sound knot.

Rotten Knot.—A rotten knot is one not as hard as the wood surrounding it.

Pin Knot.—A pin knot is a sound knot not over $\frac{1}{2}$ in. in diameter.

Standard Knot.—A standard knot is a sound knot not over $1\frac{1}{2}$ in. in diameter.

Large Knot.—A large knot is a sound knot, more than $1\frac{1}{2}$ in. in diameter.

Round Knot.—A round knot is one which is oval or circular in form.

Spike Knot.—A spike knot is one sawn in a lengthwise direction. The mean or average diameter shall be taken as the size of these knots.

Pitch Pockets.—Pitch pockets are openings between the grain of the wood, containing more or less pitch or bark. These shall be classified as small, standard and large pitch pockets.

Small Pitch Pockets.—A small pitch pocket is one not over $\frac{1}{8}$ of an inch wide.

Standard Pitch Pocket.—A standard pitch pocket is one not over $\frac{3}{8}$ of an inch wide nor over 3 in. in length.

Large Pitch Pocket.—A large pitch pocket is one over $\frac{3}{8}$ of an inch wide, or over 3 in. in length.

Pitch Streak.—A pitch streak is a well-defined accumulation of pitch at one point in the piece. When not sufficient to develop a well-defined streak, or where the fiber between grains, that is the coarse grained fiber, usually termed "spring wood" is not saturated with pitch, it shall not be considered a defect.

Wane.—Wane is bark, or the lack of wood from any cause, on edges of timbers.

Shakes.—Shakes are splits or checks in timbers which usually cause a separation of the wood between annual rings.

Rot, Dote and Red Heart.—Any form of decay which may be evident either as a dark red discoloration not found in the sound wood, or the presence of white or red rotten spots, shall be considered as a defect.

Specifications for Timber.*—The following specifications have been adopted by the American Society for Testing Materials, and represent the latest practice. (To be applied to solid members and not to composite members.)

General Requirements.—Except as noted all timber shall be cut from sound trees and sawed standard size; close grained and solid; free from defects such as injurious ring shakes and crooked grain; unsound knots; knots in groups; decay; large pitch pockets, or other defects that will materially impair its strength.

Standard Size of Sawed Timber.—Rough timbers when sawed to standard size, shall mean that they shall not be over $\frac{1}{4}$ in. scant from actual size specified. For instance, a 12 in. x 12 in. shall measure not less than $11\frac{3}{4}$ in. x $11\frac{3}{4}$ in.

Standard Dressing of Sawed Timbers.—Standard dressing means that not more than $\frac{1}{4}$ in. shall be allowed for dressing each surface. For instance, a 12 in. x 12 in. shall after dressing four sides, not measure less than $11\frac{1}{2}$ in. x $11\frac{1}{2}$ in.

Stringers. No. 1. *Longleaf Yellow Pine and Douglas Fir.*—Shall show not less than 80 per cent of heart on each of the four sides, measured across the sides anywhere in the length of the piece; loose knots, or knots greater than $1\frac{1}{2}$ in. in diameter, will not be permitted at points within 4 inches of the edges of the piece.

No. 2. *Longleaf Yellow Pine, Shortleaf Pine, Douglas Fir and Western Hemlock.*—Shall be square edged, except it may have 1 in. wane on one corner.

* For rules and specifications for grading timber, see U. S. Dept. of Agriculture, Forest Service, Bulletin 71, "Rules and Specifications for the Grading of Lumber"; Supt. of Documents, Washington, D. C. Price, 15 cents.

Knots must not exceed in their largest diameter ¼ the width of the face of the stick in which they occur. Ring shakes extending not over ⅛ of the length of the piece are admissible.

Caps and Sills. No. 1. *Longleaf Yellow Pine and Douglas Fir.*—Shall show 85 per cent heart on each of the four sides, measured across the sides anywhere in the length of the piece; to be free from knots over 2½ in. in diameter; knots must not be in groups.

No. 2. *Longleaf and Shortleaf Yellow Pine, Douglas Fir and Western Hemlock.*—Shall be square edged, except it may have 1 in. wane on one corner, or ½ in. wane on two corners. Knots must not exceed in their largest diameter ¼ the width of the face of the stick in which they occur. Ring shakes extending not over ⅛ the length of the piece are admissible.

Posts. No. 1. *Longleaf Yellow Pine and Douglas Fir.*—Shall show not less than 75 per cent heart, measured across the face anywhere on the length of the piece; to be free from knots over 2½ in. in diameter, and must not be in groups.

No. 2. *Longleaf and Shortleaf Yellow Pine, Douglas Fir and Western Hemlock.*—Shall be square edged, except it may have 1 in. wane on one corner, or ½ in. wane on two corners. Knots must not exceed, in their largest diameter, ¼ the width of the face of the stick in which they occur. Ring shakes shall not extend over ⅛ of the length of the piece.

Longitudinal Struts or Girts. No. 1. *Longleaf Yellow Pine and Douglas Fir.*—Shall show one face all heart; the other face and two sides shall show not less than 85 per cent heart, measured across the face or side anywhere in the piece; to be free from knots 1½ in. in diameter and over.

No. 2. *Longleaf and Shortleaf Yellow Pine, Douglas Fir and Western Hemlock.*—Shall be square edged and sound; to be free from knots 1½ in. in diameter and over.

Longitudinal X Braces, Sash Braces and Sway Braces. No. 1. *Longleaf Yellow Pine and Douglas Fir.*—Shall show not less than 80 per cent heart on two faces and four square edges; to be free from knots over 1½ in. in diameter.

No. 2. *Longleaf and Shortleaf Yellow Pine, Douglas Fir and Western Hemlock.*—Shall be square edged and sound; to be free from knots 2½ in. in diameter and over.

Names for Timber.*—The following description of structural timbers has been adopted by the American Society for Testing Materials:

1. *Southern Yellow Pine.*—Under this heading two classes of timber are used, (*a*) Longleaf Pine, (*b*) Shortleaf Pine.

It is understood that these two terms are descriptive of quality, rather than of botanical species. Thus, shortleaf pine would cover such species as are now known as North Carolina pine, loblolly pine and shortleaf pine. "Longleaf Pine" is descriptive of quality, and if Cuban, shortleaf, or loblolly pine is

* For a key to woods and a valuable discussion of the properties of timber, see U. S. Dept. of Agriculture, Division of Forestry, Bulletin 10, "Timber"; Supt. of Documents, Washington, D. C. Price, 10 cents.

grown under such conditions that it produces a large percentage of hard summer wood, so as to be equivalent to the wood produced by the true longleaf, it would be covered by the term "Longleaf Pine."

2. *Douglas Fir.*—The term "Douglas Fir" to cover the timber known likewise as yellow fir, red fir, western fir, Washington fir, Oregon or Puget Sound fir or pine, northwest and west coast fir.

3. *Norway Pine,* to cover what is known also as "Red Pine."

4. *Hemlock,* to cover southern or eastern hemlock; that is, hemlock from all States east of and including Minnesota.

5. *Western Hemlock,* to cover hemlock from the Pacific coast.

6. *Spruce,* to cover eastern spruce; that is, the spruce timber coming from points east of Minnesota.

7. *Western Spruce,* to cover the spruce timber from the Pacific coast.

8. *White Pine,* to cover the timber which has hitherto been known as white pine, from Maine, Michigan, Wisconsin and Minnesota.

9. *Idaho White Pine,* the variety of white pine from western Montana, northern Idaho, and eastern Washington.

10. *Western Pine,* to cover the timber sold as white pine coming from Arizona, California, New Mexico, Colorado, Oregon and Washington. This is the timber sometimes known as "Western Yellow Pine," or "Ponderosa Pine," or "California White Pine," or "Western White Pine."

11. *Western Larch,* to cover the species of larch or tamarack from the Rocky Mountain and Pacific coast regions.

12. *Tamarack,* to cover the timber known as "Tamarack," or "Eastern Tamarack," from States east of and including Minnesota.

13. *Redwood,* to include the California wood usually known by that name.

PILING.—The following specifications for oak piles have been adopted by the American Railway Engineering and Maintenance of Way Association and represent the best practice for railway purposes. Piling for highway bridges may be of smaller size, as specified.

Piling. General Requirements.—All piling shall be cut from sound live trees of slow growth, firm grain, free from ring shakes, decay, large, unsound knots, or other defects that will impair their strength and durability. They shall be butt cut, above the ground swell, and be uniformly tapering from the butt to the tip. They shall be so straight that a line stretched from the center of the pile at the butt to the center of the pile at the tip will not leave the center of the pile at any point more than two (2) in. for piles 20 ft., four (4) in. for piles 30 ft., six (6) in. for piles 40 ft., and eight (8) in. for piles 50 ft. in length. No short bends shall be allowed. All knots shall be trimmed close to the body of the pile, and the bark peeled.

White, Post or Burr Oak.—Round piles shall be not less than 12 in. diameter, 6 ft. from the butt, and not less than 10 in. diameter at the tip for piles under 30 ft. long, nor less than 9 in. diameter at the tip for piles 30 to 39 ft. long, and not less than 8 in. diameter at the tip for piles 40 ft. long and over.

Square piles shall be of timber squared throughout the entire length, smoothly hewed; they shall be not less than 14 in. nor more than 16 in. square

at the butt and not less than 10 in. square at the tip for piles under 30 ft. long, 9 in. square for piles 30 to 39 ft. long, and not less than 8 in. square for piles 40 ft. long and over. They shall show not less than 75 per cent heart.

SPECIFICATIONS FOR IRON AND STEEL.—For specifications for wrought iron, steel and cast iron, see Appendix I. The specifications for details adopted by the American Railway Engineering and Maintenance of Way Association are as follows:

Specifications for Details. *Bolts.*—Bolts shall be of wrought iron or steel, made with square heads, standard size, the length of thread to be 2½ times the diameter of bolt. The nuts shall be made square, standard size, with thread fitting closely the thread of bolt. All threads shall be cut according to U. S. standards.

Drift Bolts.—Drift bolts shall be of wrought iron or steel, with or without square head, pointed or without point, as may be called for on the plans.

Spikes.—Spikes shall be of wrought iron or steel, square or round, as called for on the plans; steel wire spikes, when used for spiking planking, shall not be used in length more than 6 in.; if greater lengths are required, wrought or steel spikes shall be used.

Packing Spools or Separators.—Packing spools or separators shall be of cast iron, made to size and shape called for on plans; the diameter of hole shall be ⅛ in. larger than diameter of packing bolts.

Cast Washers.—Cast washers shall be of cast iron. The diameter shall be not less than 3½ times the diameter of bolt for which it is used, and its thickness equal to the diameter of bolt; the diameter of hole shall be ⅛ in. larger than the diameter of the bolt.

Wrought Washers.—Wrought washers shall be of wrought iron or steel; the diameter shall be not less than 3½ times the diameter of bolt for which it is used, and not less than ¼ in. thick. The hole shall be ⅛ in. larger than the diameter of the bolt.

Special Castings.—Special castings shall be made true to pattern, without wind, free from flaws and excessive shrinkage; size and shape to be as called for by the plans.

ALLOWABLE STRESSES IN TIMBER.—The timber for highway bridges should be proportioned for the following unit stresses, given in pounds per square inch. Impact stresses are to be included as specified in § 36, Appendix I.

TABLE LXIV.

Kind of Timber.	Transverse Loading.	End Bearing	Columns Under 10 Diameters. C	Bearing Across Fiber.	Shear Along Fiber.
White Oak	1,200	1,200	1,000	500	200
Long Leaf Yellow Pine	1,500	1,500	1,000	350	100
White Pine and Spruce	1,000	1,000	600	200	100
Hemlock	800	800	500	200	100

Columns may be used with a length not exceeding 45 times the least dimension. The unit stress for lengths of more than 10 times the least dimension shall be reduced by the following formula:

$$P = C - \frac{C \cdot l}{100 d}$$

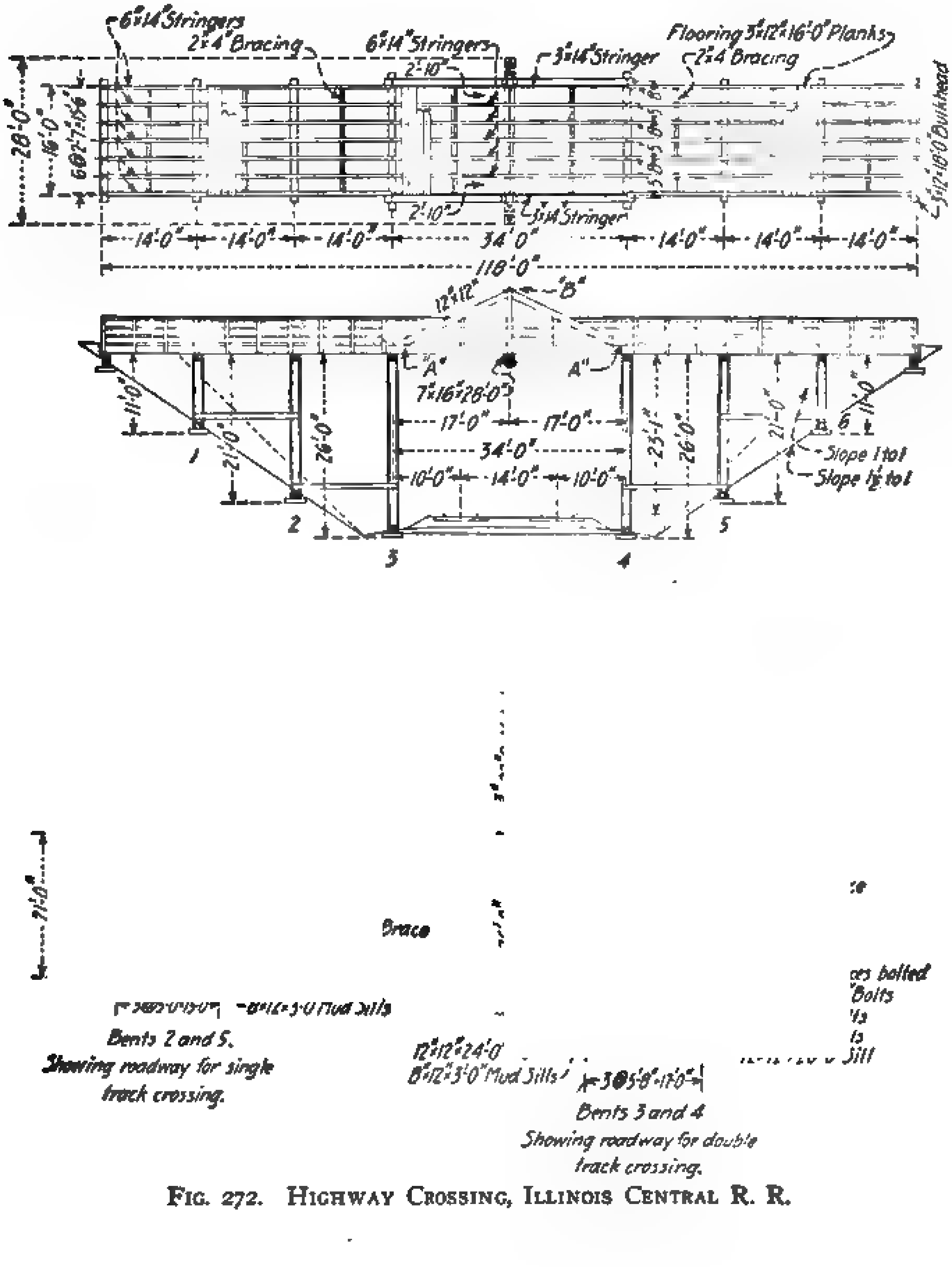


FIG. 272. HIGHWAY CROSSING, ILLINOIS CENTRAL R. R.

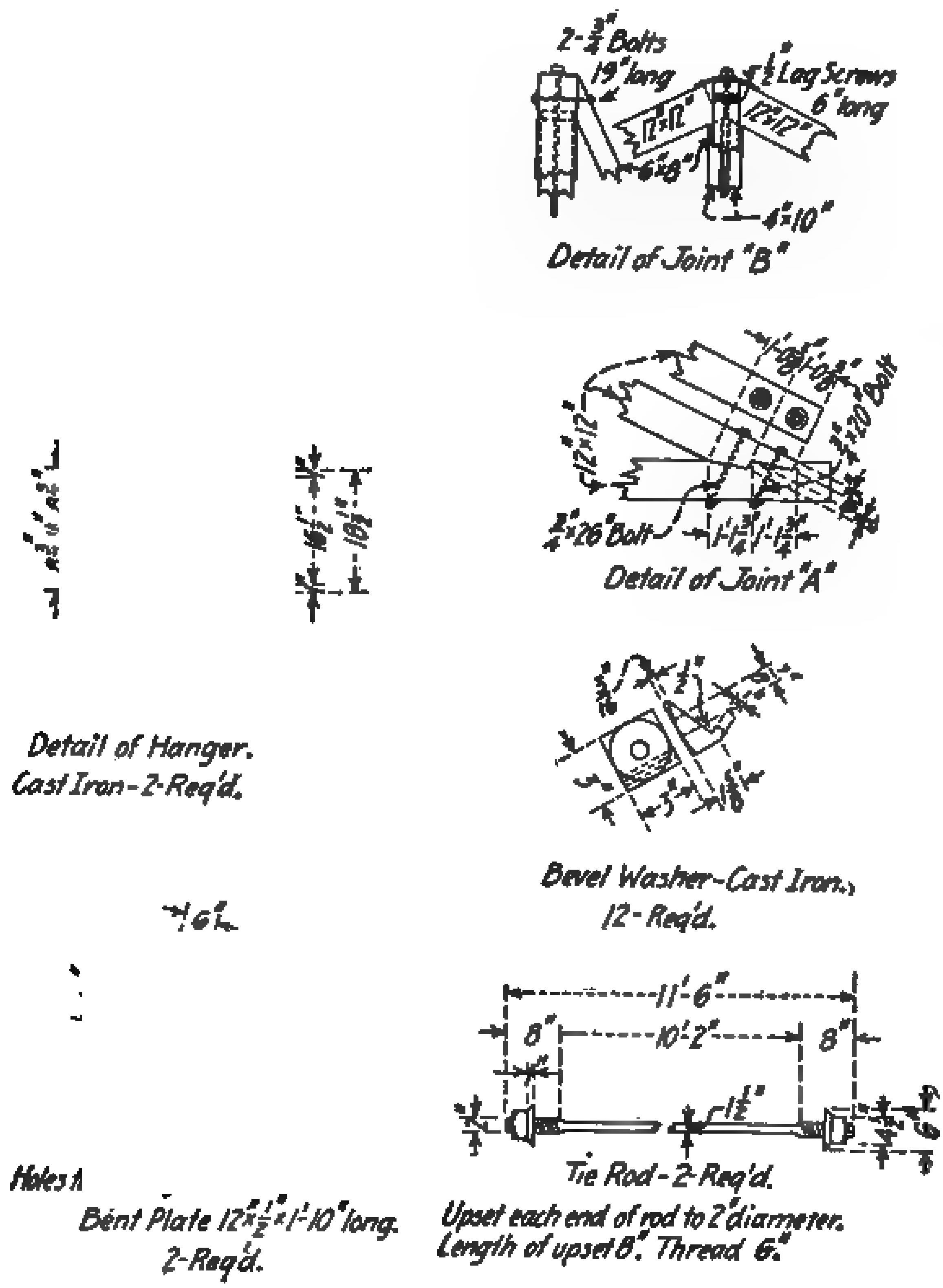


FIG. 273. DETAILS OF TIMBER TRUSS SHOWN IN FIG. 272.

where C = unit stress, as given above for short columns;

P = allowable unit stress, in lbs. per sq. in.;

l = length of column, in inches;

d = least side of column, in inches.

In applying compression formulas to columns made up of several pieces bolted together at intervals, each piece shall be assumed as an independent column.

Examples of Timber Trestles.—A standard frame bent trestle crossing for a double track railroad, as designed by the Illinois Central R. R. is shown in Fig. 272 and in Fig. 273. The floor system is heavier than is required for ordinary loads.

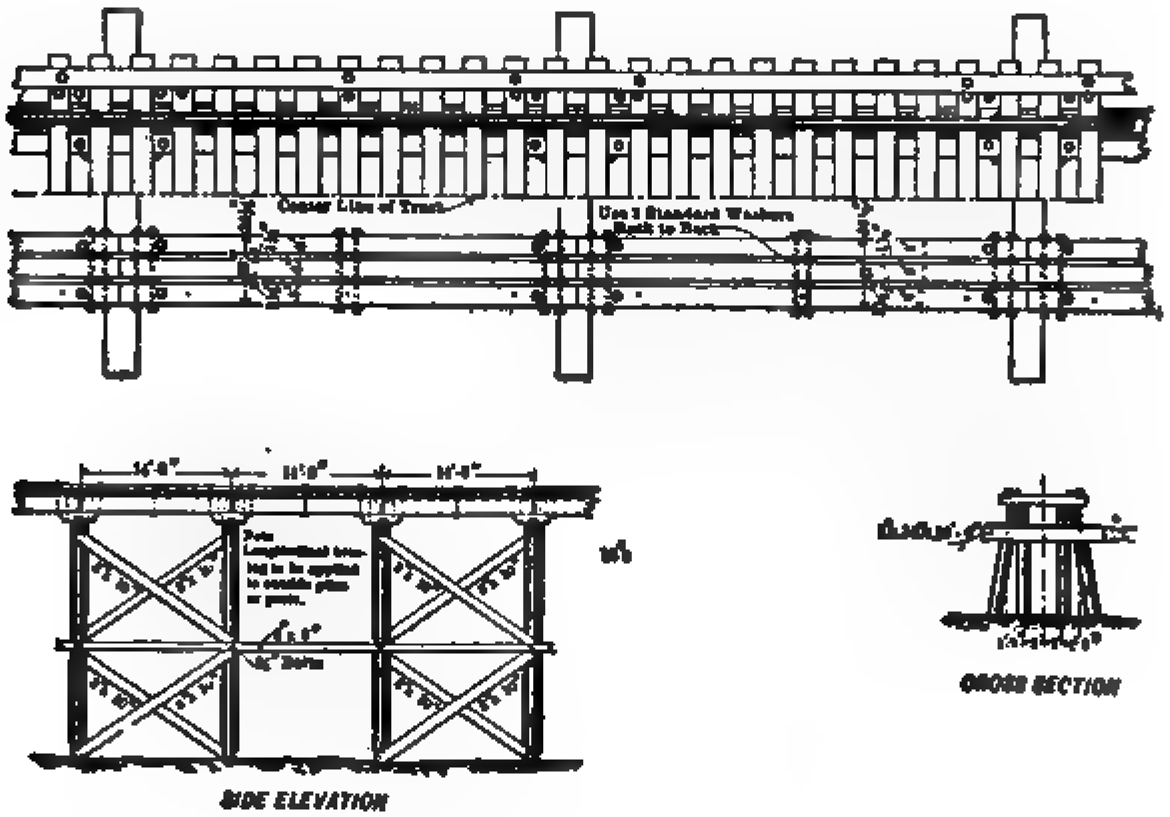


FIG. 274. PILE TRESTLE, PENNSYLVANIA LINES WEST OF PITTSBURG.

A standard pile trestle, designed by the Pennsylvania Lines West of Pittsburg, is shown in Fig. 274, while a standard framed trestle, designed by the same company, is shown in Fig. 275.

TIMBER KING POST BRIDGE.—A timber King post bridge of 34-ft. span is shown in Fig. 272, and the details of the joints are shown in Fig. 273. The chords are 12″ × 12″ with joints as at *A* and *B* in Fig. 273. A bent plate is used at joint *B*. The floorbeam consists of 2-7″ × 16″ timbers and is supported on the hanger casting.

TIMBER HOWE TRUSS BRIDGES.—The Howe truss is built with timber top and bottom chords and inclined webs, iron or steel vertical rods, and cast iron angle blocks, as shown in Fig. 276. The design of a Howe truss is principally the design of the joints and the splices. Care should be used to give adequate bearing area at right angles to the grain, and to provide against shearing along the grain of the wood. The allowable stresses for timber as given in Table LXIV should be

used. The allowable tensile stress in iron should be taken at 12,000 lbs. per square inch, while the allowable tensile stress in steel should be taken at 15,000 lbs. per sq. in.

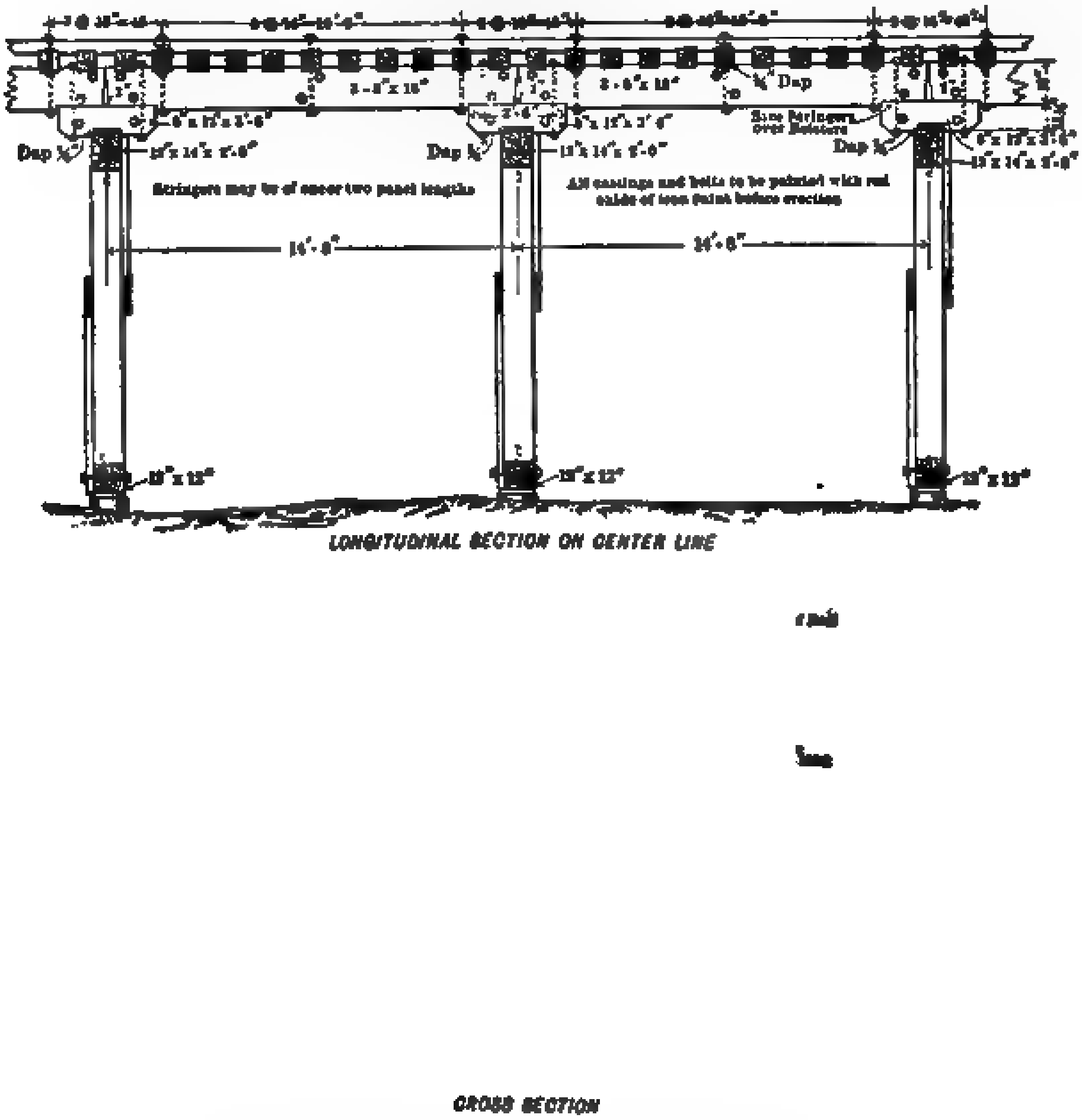


FIG. 275. FRAMED TRESTLE, PENNSYLVANIA LINES WEST OF PITTSBURG.

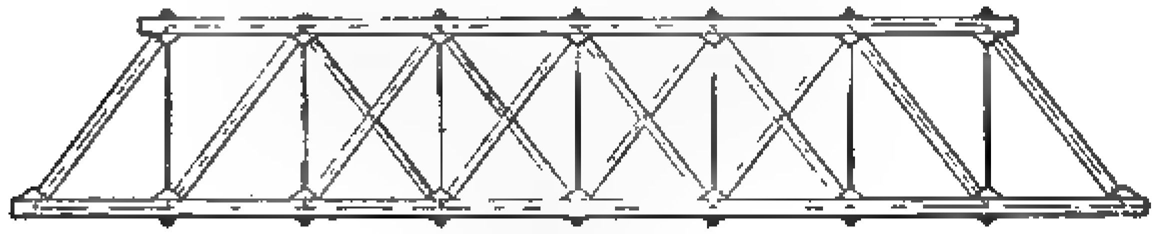


FIG. 276. THROUGH HOWE TRUSS.

A timber Howe deck electric railway bridge is shown in Fig. 277. Timber packing blocks were used, and hard wood keys were used in splicing the lower chords. The upper and lower lateral systems are of the Howe truss type.

COMBINATION BRIDGES.—In a combination bridge the top chords and the intermediate posts are made of timber, while the tension members are made of iron or steel. Combination bridges are usually made of the Pratt type. There are numerous types of combination

FIG. 277. DECK HOWE TRUSS ELECTRIC RAILWAY BRIDGE.

bridges, depending on the details of the joints, Fig. 278 and Fig. 279. A three-panel combination bridge, as designed by the Gillette-Herzog Mfg. Co., is shown in Fig. 280. A hip casting was used as shown. The shoe is made of a 12″ × 12″ × ½″ plate to which is riveted a 3½″ × 5″ × ½″ angle 12″ long. The other details are clearly shown.

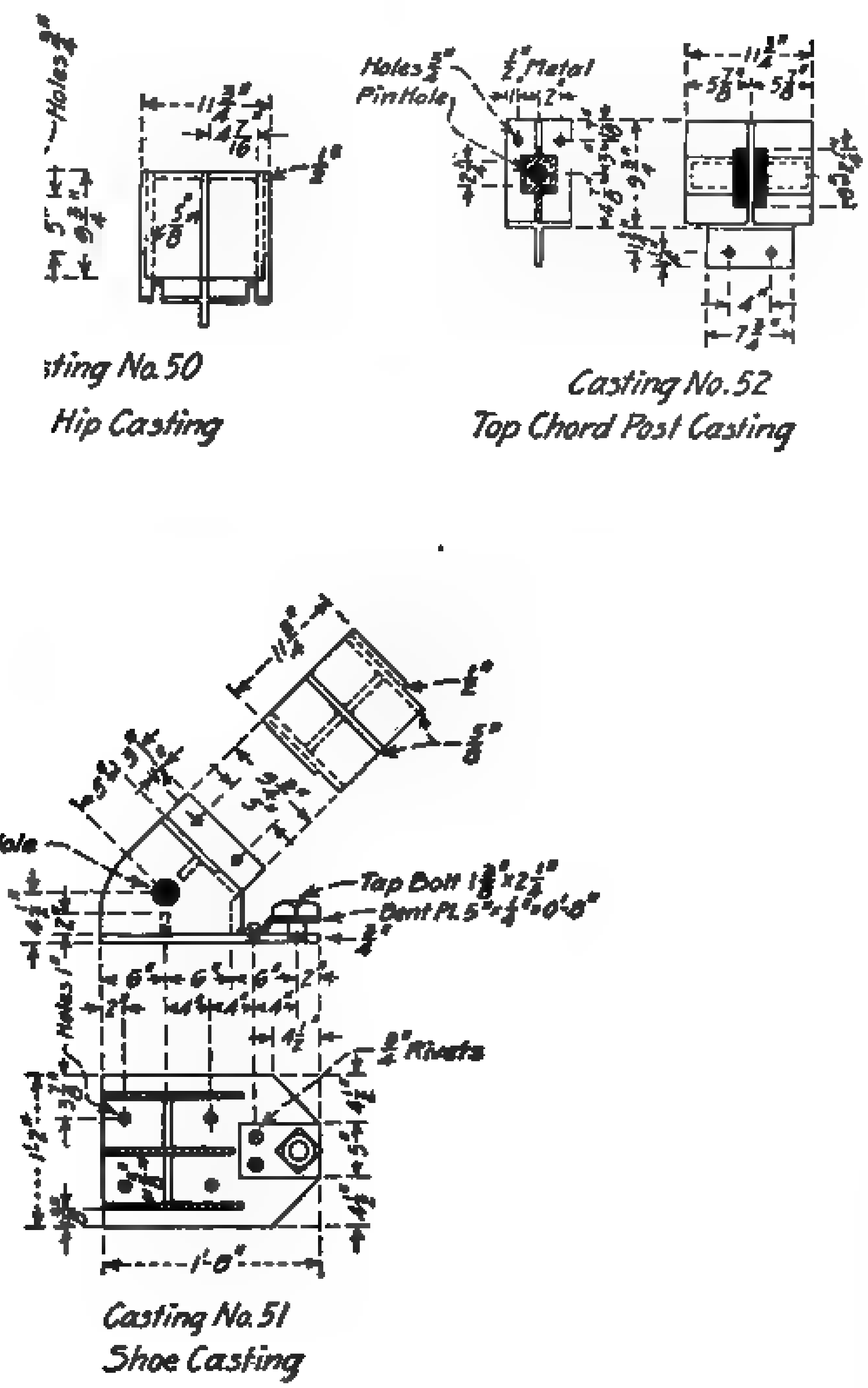


FIG. 278. CAST IRON DETAILS FOR COMBINATION BRIDGES. (PEPPARD DETAILS.)

The working plans of a 100-ft. combination highway bridge are shown in Fig. 281. Castings are used for the shoe, hip and upper chord joints, while steel shoes are used for the intermediate posts. This

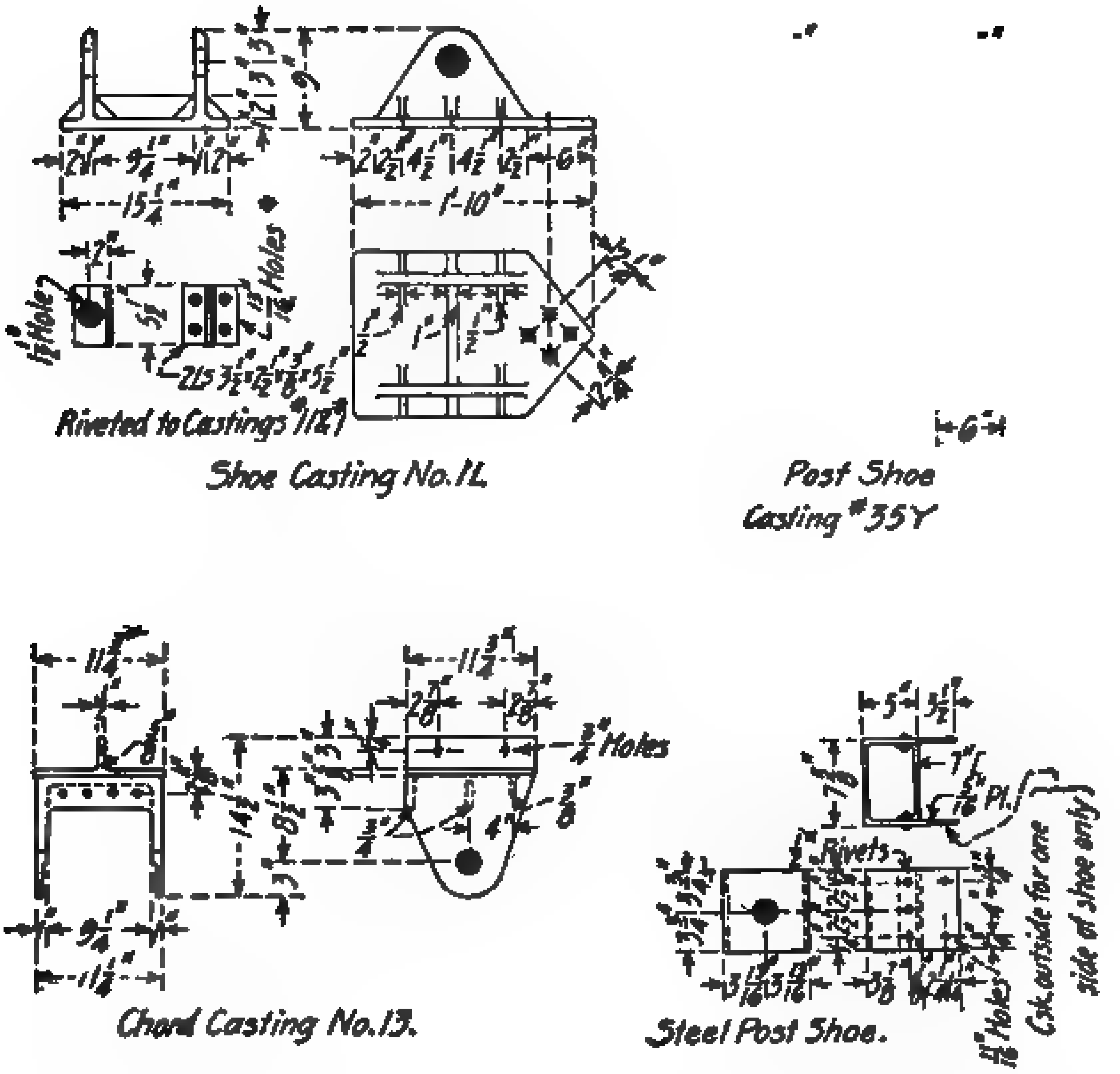


FIG. 279. DETAILS FOR COMBINATION BRIDGES. (GILLETTE-HERZOG DETAILS.)

bridge may also be built with steel floorbeams. Details of standard shoe, hip and top chord castings are shown in Fig. 278. The Gillette-Herzog Mfg. Co. standard details for combination bridges are shown in Fig. 279. It will be seen that these details give true pin joints, but they are materially heavier than the details shown in Fig. 278.

Weight of Combination Bridges.—The weight of the steel and iron in a combination bridge varies with the style of details. The weight of the steel and cast iron in bridges of the "Peppard" type are approximately 40 per cent of the weight of steel bridges of the same span, loading and dimensions. The weight of the steel and cast iron in bridges of the Gillette-Herzog Mfg. Co. type are approximately 45 to 50 per cent of the weight of steel bridges of the same span, loading and dimensions.

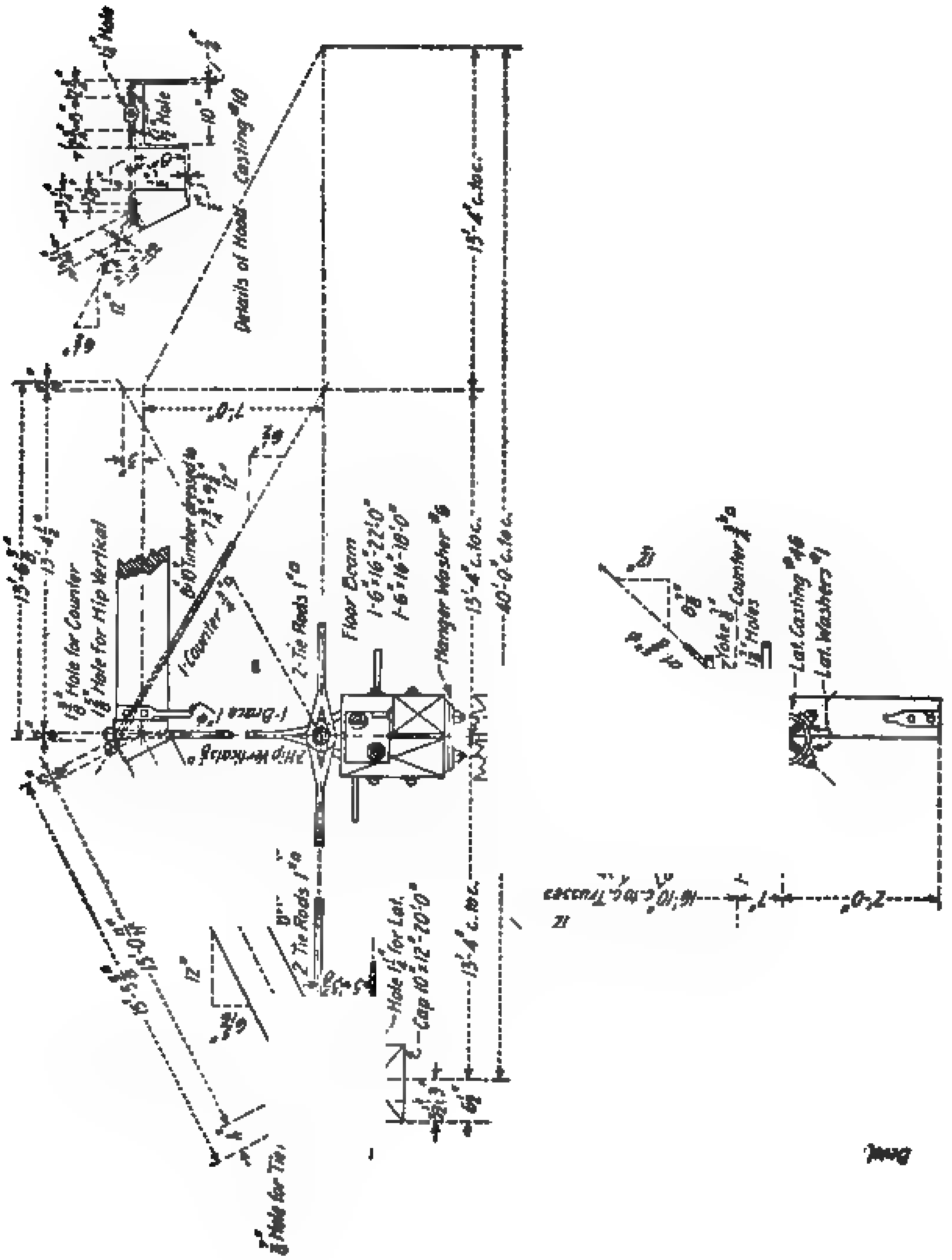


FIG. 280. THREE PANEL COMBINATION HIGHWAY BRIDGE.

CHAPTER XIX.

Erection, Estimates of Weights and Costs of Highway Bridges.

ERECTION OF STEEL HIGHWAY BRIDGES.—The details of the operation of erecting steel highway bridges will depend upon the type of bridge, length of span and character of the crossing. Short span plate girder and riveted truss bridges may be riveted or bolted up on the bank, and then swung in place by means of a gin pole (a long pole held solidly at the bottom and held in place at the top by guy ropes; the load is lifted by blocks and falls fastened to the top and bottom of the pole, while the load is swung into place by manipulating the guy ropes). Pin-connected bridges of all spans and long span riveted truss bridges are erected on falsework, usually constructed of timber. The erection of a long span pin-connected truss highway bridge will now be described.

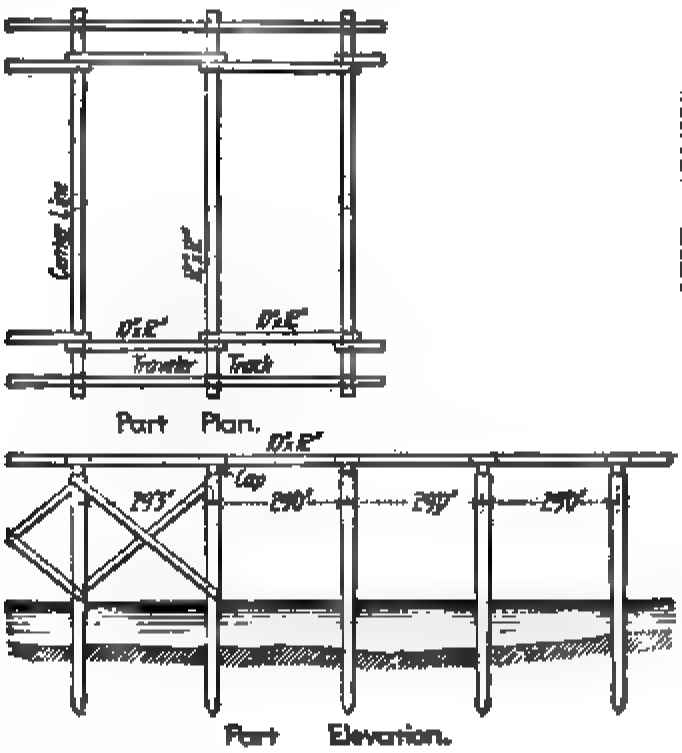


FIG. 282. FALSEWORK FOR ERECTING A HIGHWAY BRIDGE.

Erection of Petit Truss Bridge.—The falsework used in erecting a 406-ft. span Petit truss highway bridge, shown in Fig. 167 and Fig. 168, is given in Fig. 282; while the traveler used in erecting the bridge

is shown in Fig. 283. The falsework was constructed by driving 5 lines of piles to a good refusal. The piles were sawed off and capped with 12″ × 12″ timbers, as shown. The longitudinal sills for the traveler were 10″ × 12″ timbers, while 3″ × 10″ timbers were used for

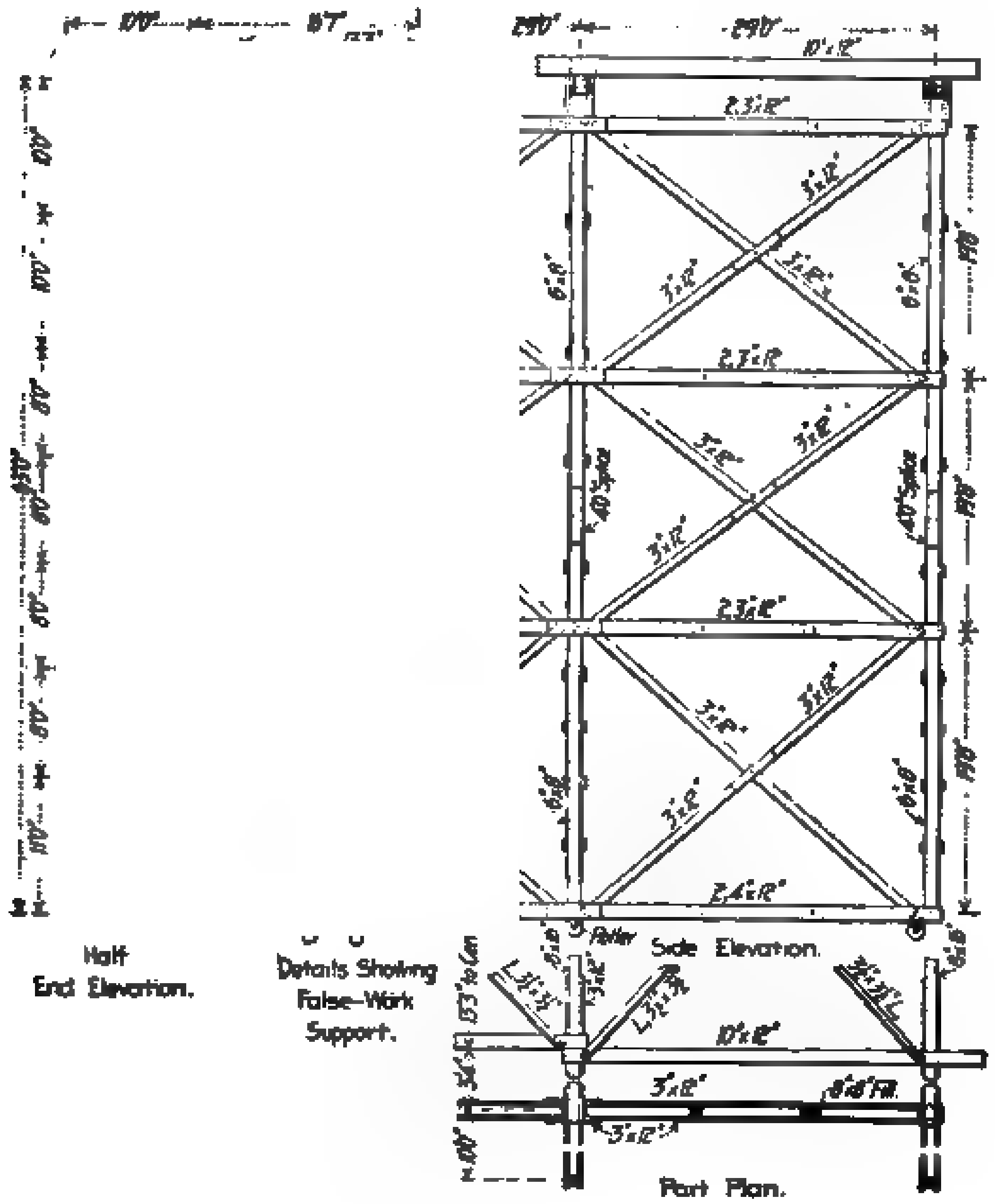


Fig. 283. Traveler for Erecting a Highway Bridge.

transverse and longitudinal bracing. The pile bents were spaced 29 ft. centers, which was the same as the panel length of the bridge. The traveler was built so that it would clear the bridge when it was erected, and was 58 ft. long. After the falsework was completed the traveler was erected and moved out on the track to the center of the bridge. The floorbeams and lower chord bars were then put in place on the falsework, care being used to see that the pedestals would come in their proper places.

The four vertical posts forming the middle double panels were then lifted into place by means of a hoisting engine and were bolted to the floorbeams. The diagonals were then put in place and the posts, diagonal and lower chords were connected up. The middle sections of the top chord were then put in place and the diagonals, top struts, sway and lateral bracing were put in place. The middle panel of the bridge was now self-supporting. The traveler was then moved 58 ft. toward one end of the bridge and the next double panel was erected, and so on until finally the end-posts were erected. The traveler was then taken back past the center of the bridge and the other end was erected in the same manner. The blocking was then knocked from under the panel points and the span was swung free. After this the riveting was completed, the floor joists and floor covering were put in place and the bridge was painted.

Pilot points and driving nuts, as shown in Fig. 183, are used in driving chord pins to protect the threads.

In erecting deck bridges the traveler is often run on the completed part of the span. Steel trestles may be erected from a traveler run on top of the completed structure; or the bents may be riveted up on the ground and then erected by using a gin pole, and after the towers have been erected the girders are raised in place by means of gin poles fastened to the tops of the towers.

In erecting small highway bridges of, say, 100-ft. span, a traveler is not ordinarily used. After building the falsework, as previously described, the four vertical posts near the center, together with the middle sections of the top chord, are raised by means of gin poles, a hand crab being used in place of a hoisting engine.

The top chords of bridges should be designed with special reference to the methods used in erecting the bridge. In bridges with parallel chords, the middle section of the top chord should be detailed so that the middle panel of both chords may be erected and made self-supporting. Splices in top chords should be placed as near panel points as practical, and between the panel point and the nearest end of the bridge. In bridges with inclined chords no splices are required, the stresses in the chords being transferred directly through the pin.

ESTIMATE OF WEIGHT OF STEEL HIGHWAY BRIDGES.—The methods of calculating the weights of bridges are: (1) estimate from finished shop drawings; (2) estimate from detail drawing; (3) estimate from stress sheet.

(1) **Estimate from Shop Drawings.**—The weight of a 160-ft. span steel highway bridge is calculated from shop drawings in Table LXXII, Chapter XXI. The weights of the members in the order, end-posts, top chords, etc., are calculated in detail. The "main members" are those that are given on the stress sheet and are either members in which stresses occur or which are specified by the designing engineer; while the "details" are plates, angles, rivets, etc., which are necessary to develop the strength of the main members. The values given in column 8 are the weights of "details" in per cent of weights of "main members." The weights per foot given in column 6 were obtained from the Cambria, Carnegie or other handbook. The weights of rivet heads were taken from Cambria Steel, and are larger than given by Carnegie. The actual shipping weight is desired, and the weights of rivet heads, only, are calculated, it being assumed that the remainder of the rivets fill the holes punched in the members. The total weights of the different parts of the bridge and the percentage of details are shown in Table LXXIII. The total weight of details in per cent of main members, exclusive of fence, joists, wall plates, bolts for lumber and spikes, is 28.7 per cent. The total weight of rivet heads in per cent of total weight of bridge, exclusive of fence, joists, etc., for three bridges are given in Table LXXV. The weights of lateral systems, lumber and other data are also given in Table LXXV.

The weight of pins in highway bridges varies from 2 to 3 per cent of the total weight of the metal, exclusive of joists, fence, etc. The weight of rivet heads in pin-connected bridges varies from 2 to 4½ per cent of the total weight of the metal, exclusive of joists, fence, etc. The weight of rivet heads in riveted highway bridges varies from 2.5 to 5 per cent of the total weight of the metal, exclusive of fence, joists, etc. The total weight of the rivet heads in the 111 ft. 6 in. riveted Pratt truss highway bridge, shown in Figs. 162 and 163, was 2.8 per cent of the total weight of the metal, exclusive of the fence, joists, etc.

The details in per cent of total weight of main members, exclusive of joists, fence, etc., will vary from 25 to 35 per cent for pin-connected and riveted highway bridges in which the top chords are made of two channels and one plate, of two angles placed back to back, or two angles and a plate; while for bridges with open chords composed of two channels laced, or two angles laced, the details will vary from 30 to 45 per cent of the weight of the main members. The details of low truss riveted highway bridges of the types shown in Figs. 146 and 147,

will weigh from 25 to 30 per cent of the total weight of main members, exclusive of fence, joists, etc.

The weight of pedestals in terms of the total weight of metal, exclusive of joists, fence, etc., will vary from 1 to 3 per cent. The weights of the details in 6 pin-connected highway bridges are given in Table LXV, while the weight of the details of the members of a 111 ft.

TABLE LXV.

DETAILS OF STEEL HIGHWAY BRIDGES.

PARTS OF BRIDGE.	160' × 16', 8 PANEL, PARALLEL CHORD.	160' × 16', 9 PANEL, INCLINED CHORD.	160' × 14', 10 PANEL, INCLINED CHORD.	119' × 16', 7 PANEL, PARALLEL CHORD.	110' × 18', 7 PANEL, PARALLEL CHORD.	135' × 18', 9 PANEL, INCLINED CHORD.
Weight of pins in per cent of total weight of metal exclusive of fence, joists, etc.	1.87	2.69	1.93	2.60	2.16	2.42
Weight of rivet heads in per cent of total weight of metal exclusive of fence, joists, etc.	3.45	4.30	3.36	2.38	2.00	2.62
Total weight of details in per cent of total weight of metal exclusive of fence, joists, etc.	28.7	33.1	25.7	26.0	30.0	28.7
Weight of pedestals in per cent of total weight of metal exclusive of fence, joists, etc.	1.45	2.10	1.70	1.30	2.10	2.45

6 in. riveted Pratt highway bridge are given in Table I. The shop drawings of this bridge are shown in Figs. 162 and 163. This bridge has exceptionally heavy details. As a rule, highway bridges do not have sufficient details to properly develop the strengths of the members. In this connection it should be remembered that the per cent of details should be larger for light country highway bridges than for heavy city or electric railway bridges designed under similar specifications.

The per cent of the details for different individual members of a bridge can be seen by the study of Table I and the estimate in Chapter XXI. It will be seen that there is a great variation in the details depending upon the make-up of the member and other conditions. The estimater should work out numerous problems and in this manner develop his estimating sense.

(2) **Estimate from Detail Drawings.**—Detail drawings show the main members partially detailed. The drawings give the number and approximate sizes of plates, the sizes of lacing bars, rivets, etc., and the approximate rivet spacing. In making an estimate from detail drawings the main members are taken from the drawings, while part of the details are supplied by the estimater. In order that the estimate be accurate the estimater must be familiar with the shop standards of the company that will fabricate the structure.

(3) **Estimate from the Stress Sheet.**—In this method the weights of the main members are calculated directly from the stress sheet, while the weights of the details are supplied by the estimater. The weight of the details may be estimated (*a*) by adding a percentage to each member—end-post, top chord, etc., or (*b*) by adding a percentage to the total weight of main members, exclusive of fence, joists, etc. The second method is very satisfactory where a standard type of bridge is used, while the first method should always be used for new types of construction.

Approximate estimates may be obtained from calculated weights, as shown in Chapter II. This method is quite accurate when the tables or diagrams have been calculated for the standards in use.

Accuracy of Estimates.—The rolls used in rolling sections are designed to give a section of the required weight when the rolls are new, so that sections are usually slightly heavier than the figured weights due to the wear or the spreading of the rolls. The allowable overweights of sections and plates are given in the Specifications in Appendix I. It is commonly specified that the actual weight of fabricated steel work may vary not more than 2½ per cent from the figured weight. This means that where fabricated structural steel is bought at a pound price, the purchaser will have to pay for the actual weight, providing it does not exceed the calculated weight by more than 2½ per cent. Where fabricated structural steel work is more than 2½ per cent lighter than the calculated weight, the purchaser may refuse to accept the material. This latter case never occurs unless sections lighter than those shown on the drawings are substituted. The estimate made from shop drawings should be used as a basis for comparison. The results obtained from the detail drawings or from stress sheets should not vary from shipping weight by more than 1½ to 2 per cent, and should be a little heavy rather than light. Estimates from stress sheets should be made only by a skilled estimater.

Shop Waste.—The shipping weight of fabricated structural steel will be less than the weight of the rolled steel, due to the loss in rivet slugs, clippings, beveled cuts, milling, etc. This loss will vary from 3 to 5 per cent for highway bridges.

Estimate of Lumber.—Lumber is estimated in board feet, a board foot being a piece 12 in. square and 1 in. thick. Commercial sizes of lumber are less than the stated dimensions, so that where full sized timbers are desired it is necessary to specify this explicitly. Specifications for bridge timbers are given in Chapter XVIII.

Weight of Floor.—The weight of oak is commonly taken as $4\frac{1}{2}$ and pine $3\frac{1}{2}$ lbs. per foot B. M. The actual weights of other materials should be calculated, see Chapter II and Appendix I.

Miscellaneous Data.—The American Bridge Company's standards contain the following data: For low spans the center to center length should equal the clear span plus 1 ft. 6 in.; while the length over all is equal to the *c* to *c* length plus 1 ft. For high trusses the center to center length should equal the clear span plus 2 ft.; while the length over all is equal to the *c* to *c* length plus 1 ft. 6 in.

Standard lattice rail is made of two angles $2\frac{1}{2}'' \times 2'' \times \frac{3}{16}''$, 18″ back to back, with double lacing made of $1\frac{1}{2}'' \times \frac{3}{16}''$, 12″ center to center. Total weight of this rail is $9\frac{1}{2}$ lbs. per lineal foot, plus 25 lbs. for each end. This weight does not include the posts. Posts for gas pipe rail weigh 25 lbs. each and should be placed at each panel point and midway point.

Anchor bolts for high spans may be estimated at 20 lbs. per span. Anchor bolts for low truss spans may be estimated at 16 lbs. per span. Floor bolts through wheel guard weigh 1 lb. per lineal foot of span.

In estimating the weight of sidewalk brackets run the floorbeam out a distance equal to the clear width of the sidewalk and add 100 lbs. for the weight of the railing post.

ESTIMATE OF COST.—The cost of a steel highway bridge may be divided into (1) cost of material, (2) cost of fabrication, (3) cost of transportation, (4) cost of erection, (5) cost of substructure, and (6) profit. The subject of costs is a very difficult matter to handle, and the author would caution the reader to use the data given on the following pages with great care, for the reason that costs are always relative and what may be a fair cost in one case may be sadly in error in another case, which appears to be an exact parallel. The price of labor will be given in each case or the cost will be charged on the basis

of 40 cents per hour, which includes labor, cost of management, tools. The costs given below are the average costs for a shop with a capacity of about 1,000 tons per month that has made a specialty of highway bridge work. The costs given are based on a charge of 40 cents per hour for the number of hours actually consumed in getting out the contract. This charge is assumed to cover the cost of management, cost of operation and maintenance, as well as the cost of labor. The cost of management in a small shop is very low, but in a large concern it may amount to as much as 35 to 40 per cent of all the other charges combined. For this reason small shops can often fabricate light highway bridge steel for a less cost than the large shops. The prices given are about an average of those used by the agents of the company above, and have been checked against actual costs for the greater part. For additional data on the costs of structural steel, see the author's "The Design of Steel Mill Buildings" and "The Design of Walls, Bins and Grain Elevators."

Cost of Material.—The price of structural steel is quoted in cents per pound delivered free on board cars (f. o. b.) at the point at which quotation is made. Current prices may be obtained from the Engineering News, Iron Age, or other technical papers. Present prices (1908) f. o. b. Pittsburg, Pa., are as follows:

TABLE LXVI.

PRICES OF STRUCTURAL STEEL (1908), F. O. B. PITTSBURG, PA., IN CENTS PER POUND.

Material.	Price in Cts. Per Lb.
I beams 18″ and over	1.70
I beams and channels 3″ to 15″	1.60
Angles 3″ to 6″ inclusive	1.70
Angles over 6″	1.80
Tees 3″ and over	1.80
Zees 3″ and over	1.70
Channels, angles, T's and Z's under 3″	1.60
Plates, structural, base	1.60
Plates, flange, base	1.70
Bars and rivet rods	1.60
Deck beams and bulb angles	1.90
Checkered plates	2.25
Forged rounds 5″ to 11″ diameter	2.75
Eye-bar flats 8″ to 12″ inclusive	2.10
Eye-bars over 6″ and under 8″	1.90
Eye-bars 6″ and under	1.60

Eye-bars over 12″ wide subject to special arrangement.
Rolled rounds over 3″ diameter, 18″ long or over, 0.35 cents per lb. extra.
Rolled rounds over 3″ diameter, under 18″ long, 0.65 cents per lb. extra.

The prices above are net with the exception of those for plates and bars which are subject to standard extras as follows:

Extras.—*Shapes, Plates and Bars:*

(Cutting to length.)	
Under 3′ to 2′, inclusive	0.25 cts. per lb.
Under 2′ to 1′, inclusive	0.50 " "
Under 1′	1.55 " "

Extras—Plates (Card of January 7, 1902):
Base ¼″ thick, 100″ wide and under, rectangular (see sketches).
Weights—see Mfgr's. Standard Specifications, Carnegie or Cambria Hand-Books.

	Per 100 Lbs.
Widths—100″ to 110″	$ .05
110″ to 115″	.10
115″ to 120″	.15
120″ to 125″	.25
125″ to 130″	.50
Over 130″	1.00
Gages under ¼″ to and including 3/16″	.10
Gages under 3/16″ to and including No. 8	.15
Gages under No. 8 to and including No. 9	.25
Gages under No. 9 to and including No. 10	.30
Gages under No. 10 to and including No. 12	.40
Complete circles	.20
Boiler and flange steel	.10
Marine and fire box	.20
Ordinary sketches	.10

(Except straight taper plates, varying not more than 4″ in width at ends, narrowest end not less than 30″, which can be supplied at base prices.)

TABLE LXVII.

STANDARD CLASSIFICATION OF EXTRAS ON IRON AND STEEL BARS.*

Rounds and Squares.

Squares up to 4½ inches only. Intermediate sizes take the next higher extra.

	Per 100 Lbs.
¾ to 3 inches	Rates.
⅝ to 11/16 "	$0.10 extra.
½ to 9/16 "	.20 "
7/16 "	.40 "
⅜ "	.50 "

* Adopted August, 1902.

$\frac{5}{16}$	inches	....	.60 extra.
$\frac{1}{4}$ and $\frac{9}{32}$	"	....	.70 "
$\frac{7}{32}$	"	....	1.00 "
$\frac{3}{16}$	"	....	2.00 "
$3\frac{1}{16}$ to $3\frac{1}{2}$	"	....	.15 "
$3\frac{9}{16}$ to 4	"	....	.25 "
$4\frac{1}{16}$ to $4\frac{1}{2}$	"	....	.30 "
$4\frac{9}{16}$ to 5	"	....	.40 "
$5\frac{1}{8}$ to $5\frac{1}{2}$	"	....	.50 "
$5\frac{5}{8}$ to 6	"	....	.75 "
$6\frac{1}{8}$ to $6\frac{1}{2}$	"	....	1.00 "
$6\frac{5}{8}$ to $7\frac{1}{4}$	"	....	1.25 "

Flat Bars and Heavy Bands.

					Per 100 Lbs.
1 to 6	inches	x $\frac{3}{8}$ to 1	inch	....	Rates.
1 to 6	"	x $\frac{1}{4}$ and $\frac{5}{16}$	"	....	$0.20 extra.
$\frac{11}{16}$ to $\frac{15}{16}$	"	x $\frac{3}{8}$ to $\frac{3}{4}$	"	....	.40 "
$\frac{11}{16}$ to $\frac{15}{16}$	"	x $\frac{1}{4}$ and $\frac{5}{16}$	"	....	.50 "
$\frac{9}{16}$ and $\frac{5}{8}$	"	x $\frac{3}{8}$ to $\frac{1}{2}$	"	....	.50 "
$\frac{9}{16}$ and $\frac{5}{8}$	"	x $\frac{1}{4}$ and $\frac{5}{16}$	"	....	.70 "
$\frac{1}{2}$	"	x $\frac{3}{8}$ and $\frac{7}{16}$	"	....	.90 "
$\frac{1}{2}$	"	x $\frac{1}{4}$ and $\frac{5}{16}$	"	....	1.10 "
$\frac{7}{16}$	"	x $\frac{3}{8}$	"	....	1.00 "
$\frac{7}{16}$	"	x $\frac{1}{4}$ and $\frac{5}{16}$	"	....	1.20 "
$\frac{3}{8}$	"	x $\frac{1}{4}$ and $\frac{5}{16}$	"	....	1.50 "
$1\frac{1}{8}$ to 6	"	x $1\frac{1}{16}$ to $1\frac{3}{16}$	"	....	.10 "
$1\frac{1}{8}$ to 6	"	x $1\frac{1}{4}$ to $1\frac{1}{2}$	"	....	.20 "
$1\frac{3}{4}$ to 6	"	x $1\frac{5}{8}$ to $2\frac{3}{4}$	"	....	.30 "
$3\frac{1}{8}$ to 6	"	x 3 to 4	"	....	.40 "

COST OF FABRICATION.—The cost of fabrication of steel bridges may be divided into (a) cost of drafting, (b) cost of mill details, and (c) cost of shop labor.

(a) **Cost of Drafting.**—The cost of drafting varies with the character of the bridge and the shop methods of the bridge company. There are two methods in common use for detailing bridges. The first method is to make the drawings so complete that templates can be made for each individual piece, separately on the bench. The second method is to give on the drawings only sufficient dimensions to locate the intersections of the members and the positions of the pieces, leaving the template maker to work out the details on the laying-out-floor. The first method is illustrated in Figs. 163 and 285 and the second in Figs. 156 and 159. Most small bridge companies use the second method, while the American Bridge Company and other large bridge

companies use the first method. The cost of making details by the second method will vary from $1.00 to $2.00 per ton; while the cost of making details by the first method varies from $2.00 to $4.00 per ton.

(b) **Cost of Mill Details.**—The American Bridge Co.'s card for cost of mill details differs somewhat from the standard card of cost of mill details given in the author's book on "The Design of Steel Mill Buildings," Chapter XXVIII.

American Bridge Co.'s card of cost of mill details:

Mill rates:

"*a*"—0.15 cts. per lb.

This covers:

Plain punching 1 size hole in web only.

Plain punching 1 size hole in one or both flanges.

"*b*"—0.25 cts. per lb.

This covers plain punching one size hole in either web and one flange or web and both flanges. (The holes in the web and flange must be of the same size.)

"*c*"—0.30 cts. per lb.

This covers:

Punching of 2 size holes in the web only.

Punching of 2 size holes in one or both flanges.

"*d*"—0.35 cts. per lb.

This covers punching and assembling into girders. Coping, ordinary beveling, including riveting and bolting of standard connection angles (this class includes beams shipped with connection angles bolted).

"*e*"—0.40 cts. per lb.

This covers the punching of one size hole in the web and another size hole in the flanges.

"*f*"—0.15 cts. per lb.

This covers cutting to length with less variation than plus or minus $\frac{1}{8}''$.

"*r*"—0.50 cts. per lb.

This covers beams with cover plates, shelf angles and ordinary riveted beam work, unless they are charged under class "*d*."

If this work consists of bending or any unusual work, the beams should not be included in the beam classification but estimated the same as riveted work. On all material estimated for cost at mill rates, 10 cts. per 100 lbs. is to be allowed for painting and 5 cts. per 100 lbs. is to be allowed for drawings.

Fittings.—All fittings, whether loose or attached, such as angle connections, bolts, separators, tie rods, etc., whenever they are estimated on in connection with beams or channels, to be charged at 1.55 cts. per lb. over and above the base price. The extra charge for painting is to be added to the price for fittings also. The base price on which fittings are based is not the base price of the beams to which they are attached, but is in all cases the base price of beams 15″ and under. The above rates will not include painting or oiling, which should be charged at the rate of 0.10 cts. per lb. for one coat, over and above the base

price plus the extra specified above. For plain punched beams, where holes of more than two sizes are used, 0.15 cts. per lb. should be added for each additional size hole; for example—Plain punched beams, where three size holes occur, would be indicated as "*e*" plus 0.15 cts.; four size holes as "*e*" plus 0.30 cts.; for example—A beam with ⅝″ and ¾″ holes in the flanges and ⅝″ and ¾″ holes in the web should be included in class "*e*."

Cutting to length can be combined with any of the other rates except "*d*" and would have to be indicated, for example—Plain punching one size hole in either web and one flange, or web and both flanges, and cutting to length would be marked "*bf*," which would establish a total charge of 0.40 cts. per lb.

Note to class "*d*":

No extra charge can be rendered to this class for punching various size holes or cutting to length; in other words, if a beam is coped, or has connection angles riveted or bolted to it, it makes no difference how many size holes are punched in this beam—the extra will always be the same, namely, 0.35 cts.

Beams with shelf angles, short seat angles or cover plates are strictly not covered by card rates. They can be charged either under class "*d*," this rate covering only the beam proper, in which case all other material ought to be rated as fittings with the charge of 1.55 cts. per. lb. over and above the base price, or they can be classified under a special shop rate, "*r*,"—0.50 cts. per lb. This rate applies to all material forming the piece. It is the intention to charge whatever figures are the lowest, in order to give the customer the benefit of the doubt. In preparing the estimate, beam material should be marked with the letter "*b*" and to this should be added the letter giving the classification, thus: A beam punched with one size hole in one or both flanges will be marked "*b a*," etc.

In ordering material from the mill the following items should be borne in mind: Where beams butt at each end against some other member, order the beams ½ inch shorter than the figured lengths; this will allow a clearance of ¼ inch if all beams come ⅜ of an inch too long. Where beams are to be built into the wall, order them in full lengths, making no allowance for clearance. Order small plates in multiple lengths. Irregular plates on which there will be considerable waste should be ordered cut to template. Mills will not make reëntrant cuts in plates. Allow ¼ of an inch for each milling for members that have to be faced. Order web plates for girders ¼ to ½ inch narrower than the distance back to back of angles. Order as nearly as possible every thing cut to required length, except where there are liable to be changes made, in which case order long lengths.

It is often possible to reduce the cost of mill details by having the mills do only part of the work, the rest being done in the field, or by sending out from the shop to be riveted on in the field connection angles and other small details that would cause the work to take a very much

higher price. Standard connections should be used wherever possible, and special work should be avoided.

The classification of iron and steel bars is given in Table LXVII. The full extra charges for sizes other than those taking the base rate are seldom enforced; one-half card extras being very common.

In estimating the cost of plain material in a finished structure the shipping weight from the structural shop is wanted. The cost of material f. o. b. the shop must therefore include the cost of waste, paint material, and the freight from the mill to the shop. The waste is variable but as an average may be taken at 4 per cent. Paint material may be taken as one dollar per ton. The cost of plain material at the shop would be

Average cost per pound f. o. b. mill, say	1.75	cts.
Add 4 per cent for waste	.07	"
Add $1.00 per ton for paint material	.005	"
Add freight from mill to shop (Pittsburg to Chicago)	.165	"
Total cost per pound f. o. b. shop	1.990	"

To obtain the average cost of steel per pound multiply the pound price of each kind of material by the percentage that this kind of material is of the whole weight, the sum of the products will be the average pound price, see pp. 424 and 449.

(c) **Cost of Shop Labor.**—The cost of shop labor may be calculated for the different parts of the bridge or the cost may be calculated for the bridge as a whole.

Cost of Individual Parts of Bridges.—The cost of fabricating joists and other similar members should be estimated on the basis of mill details, which see.

Eye-bars.—The shop cost of eye-bars varies with the size and length of the bars and the number made alike. The following costs are a fair average: Average shop costs of bars 3 inches and less in width and $\frac{3}{4}$ inch and less in thickness is from 1.20 to 1.80 cts. per lb., depending upon the length and size. A good order of bars running $2\frac{1}{2}'' \times \frac{3}{4}''$ to $3'' \times \frac{3}{4}''$, and from 16 to 20 ft. long, with few variations in size, will cost about 1.20 cts. per lb. Large bars in long lengths ordered in large quantities can be fabricated at from 0.55 to 0.75 cts. per lb. To get the total cost of eye-bars the cost of bar steel must be added to the shop cost. Half card extras given in Table LXVII should ordinarily be added to the base price of plain steel bars.

Chords, Posts and Towers.—In lots of at least four, the shop cost is about as follows: Members made of two channels and a top plate with lacing on the bottom side, or two channels laced on both sides cost about 1.00 to 0.85 cts. per lb. for pin-connected members weighing from 600 to 1,500 lbs.; and about 0.80 to 0.70 cts. per lb. for members with riveted end connections. Members made of four angles laced cost from 0.80 to 1.10 cts. per lb. for members with riveted ends. Members made of two angles battened will cost about 0.50 cts. per lb. Angles used without end connections should have their cost estimated on the basis of mill details, which see.

Pins.—The cost of chord pins will vary with the size, number and other requirements. The shop cost of chord pins and nuts may be estimated at from 2.00 to 3.00 cts. per lb. Rollers will cost practically the same as pins. Rolled rounds (pin rounds) are used for making pins and rollers.

Latticed Fence.—The shop cost of light simple latticed fence made of two 2″ × 2″ angles, with double lacing and about 18″ deep, will be about 2.00 cts. per lb.; while the shop cost of latticed fence, with ornamental rosettes or ornamental plates, may be as much as 4.00 or 5.00 cts. per lb.

Floorbeams.—Plate girders used for floorbeams will cost from 0.60 to 1.25 cts. per lb., depending upon the weight, details and number made at one time. Floorbeams made of rolled I-beams will cost from 0.50 to 0.75 cts. per lb.

Shop Costs of Structures as a Whole.—The cost will be taken up under the head of pin-connected bridges, riveted bridges, plate girder bridges, combination bridge metal, and Howe truss metal.

Shop Cost of Pin-connected Bridges.—The shop costs of pin-connected highway bridges, exclusive of fence and joists, are about as follows:

Bridges weighing	5,000 lbs. and less	1.30	cts. per lb.
" "	5,000 to 10,000 lbs	1.20	" "
" "	10,000 to 20,000 lbs	1.00	" "
" "	20,000 to 40,000 lbs	0.90	" "
" "	40,000 to 60,000 lbs	0.80	" "
" "	60,000 to 100,000 lbs	0.75	" "

These costs include detailing and one coat of shop paint. For reaming add 0.15 cts. per lb.

Shop Cost of Riveted Truss Bridges.—The shop costs of riveted truss bridges, exclusive of fence and joists, are about as follows:

Bridges weighing 5,000 lbs. and less	1.15	cts.	per	lb.
"	" 5,000 to 10,000 lbs.	1.00	"	"
"	" 10,000 to 20,000 lbs.	0.90	"	"
"	" 20,000 to 40,000 lbs.	0.85	"	"
"	" 40,000 to 60,000 lbs.	0.75	"	"
"	" 60,000 to 100,000 lbs.	0.70	"	"

These costs include detailing and one coat of shop paint. For reaming add 0.15 cts. per lb.

Shop Cost of Plate Girder Bridges.—The shop costs of plate girder highway bridges, exclusive of fence and joists, are about as follows:

Spans weighing 10,000 lbs. and less	0.90	cts.	per	lb.
" " 10,000 to 20,000 lbs	0.85	"	"	
" " 20,000 to 40,000 lbs	0.75	"	"	
" " 40,000 to 60,000 lbs	0.70	"	"	
" " 60,000 to 100,000 lbs	0.50–0.60	"	"	

The above costs include detailing and one coat of shop paint. For reaming add 0.15 cts. per lb.

Shop Cost of Tubular Piers and Culverts.—The shop costs of steel tubular pier shells and steel culvert pipe are about as follows:

Tubes 18" to 24" diameter, $\frac{1}{4}$" metal	1.00	cts.	per	lb.
" 24" to 30" diameter, $\frac{1}{4}$" to $\frac{3}{8}$" metal	0.75 to 0.65	"	"	
" 30" to 48" diameter, $\frac{1}{4}$" to $\frac{3}{8}$" metal	0.70 to 0.60	"	"	
" 48" to 72" diameter, $\frac{1}{4}$" to $\frac{3}{8}$" metal	0.65 to 0.50	"	"	

The above shop costs include detailing and one coat of shop paint. The necessary bracing and rods for tubular piers are included.

The Cost of Combination Bridge Metal.—Where the bars and rods are standard and the castings are made from standard patterns, the metal for combination bridges can be fabricated at about the same cost per lb. as for pin-connected spans weighing the same as the weight of the metal in the combination bridges.

The Shop Cost of Howe Truss Bridge Metal.—The shop cost of highway bridge castings made from standard patterns is from 1.50 to 2.00 cts. per lb. The shop cost of the plates, rods and other miscellaneous iron work will cost 2.00 to 2.50 cts. per lb.

TRANSPORTATION.—Fabricated bridge steel commonly takes a "fifth-class rate" when shipped in car load lots, and a "fourth-class rate" when shipped "local" (in less than car load lots). The minimum car load depends upon the railroad and varies from 20,000 to 30,000 lbs. Tariff sheets giving railroad rates may be obtained from any railroad company. The shipping clerk should be provided with the clearances of all tunnels and bridges on different lines so that the car may be properly loaded.

ERECTION.—The cost of erection ordinarily includes: (1) the cost of hauling the bridge to the bridge site; (2) the building of the falsework and the placing of the steel in position; (3) the riveting up of the bridge, and (4) painting the steel and the woodwork.

Hauling.—Transportation over country roads will ordinarily cost about 25 cts. per ton mile, in addition to the cost of loading and unloading. In estimating the cost of hauling on any particular job the length of haul, kind of roads, price of teams and labor, and the character of the teams should be considered. The cost of loading on the wagons and unloading will depend upon the local conditions, but will ordinarily be from 25 to 50 cts. per ton.

Falsework.—If piles are to be used the cost should be carefully estimated. The cost of the piles in place will vary with the cost of piles and local conditions. Under ordinary conditions piles in falsework will cost from 25 to 50 cts. per lineal foot in place. The cost of the timber will depend upon local conditions and upon what use is made of it after erection. The flooring plank can often be used in the falsework without serious injury. The cost of erecting the timber in the falsework will ordinarily be from $6.00 to $8.00 per thousand ft. B. M.

Erection of Tubular Piers.—The cost of setting tubular piers will depend upon the conditions. Tubes 36 inches in diameter and 20 ft. long have been set in favorable locations for $25.00 per pair, not including the driving of the piles or the placing of the concrete. It is, however, not safe to estimate the cost of setting tubes from 36 to 48 inches in diameter under even favorable conditions at less than $2.00 per lineal foot of tube. When the cost of setting tubes is estimated by weight, it should be figured at from $10.00 to $20.00 per ton, for ordinary conditions. It will commonly cost from 25 to 50 cts. per lineal ft. to drive piles in tubes, in addition to the cost of the piles, which will vary from 10 to 20 cts. per lineal foot. For methods of erecting steel tubular piers, see Chapter XV.

Placing and Bolting.—The cost of placing and bolting up riveted spans, and erecting pin-connected spans, no rivets being driven, is about as follows:

Spans from	30 to 60 ft.	$12.00 to $15.00	per ton.
"	60 to 100 ft.	10.00 to 12.00	"
"	100 to 150 ft.	9.00 to 10.00	"
"	150 ft. and up	8.00	"

The cost of driving field rivets in pin-connected spans will vary from 7 to 12 cts. per rivet, while the cost of driving rivets in riveted trusses will vary from 6 to 10 cts. per rivet. The number of rivets in riveted low trusses depends upon the number of panels and the style of details, and will be about 155 to 200 for a three-panel bridge, and 400 to 500 for a six-panel bridge. The number of rivets in a through riveted highway bridge will be about 250 to 300 for a four-panel bridge and 1,300 to 1,500 for a nine-panel bridge. Pin-connected bridges ordinarily have about ⅓ as many field rivets as a riveted bridge of similar dimensions.

Cost of Erecting Combination Bridges.—Mr. O. E. Peppard, Missoula, Montana, who has had a wide experience, gives the following as average costs for erecting combination bridges of 100- to 200-ft. span, labor being estimated at $3.00 per day of 10 hours: Falsework, 30 to 50 cts. per lineal foot of span; erecting the lumber in the span, $10.00 per thousand ft. B. M.; erecting iron and steel in the span, $10.00 per ton; erecting timber joist, $3.00 per thousand ft. B. M.; painting the timber, $2.00 per thousand ft. B. M.; painting the iron and steel, $2.00 per ton.

Mr. Peppard gives the cost of erecting a 200-ft. span combination highway bridge with 16-ft. roadway, designed for a live load of 800 lbs. per lineal foot of span, as follows:

Placing the falsework, 200 lineal foot @ 50 cts	$100.00
Erecting the timber in the span, 16,200 ft. B. M. @ $10.00	162.00
Erecting the iron and steel in span, 14.65 tons @ $10.00	146.50
Erecting joists and flooring, 16,500 ft. B. M. @ $3.00	49.50
Painting timber in trusses and fence, 16,200 ft. B. M. @ $2.00	32.40
Painting steel and iron, 14.65 tons @ $2.00	29.30
Transportation of men and tools	50.00
Depreciation and interest on tools	30.00
Total cost of erection	$599.70

COST OF PAINTING.—The amount of materials required to make a gallon of paint and the surface of steel work covered by one gallon are given in Table LXVIII. Steel bridges should be painted with one coat of linseed oil, linseed oil with lampblack filler, or red lead paint at the shop; and two coats of first-class paint after erection. The two field coats should be of different colors; care being used to see that first coat is thoroughly dry before applying the second coat. Steel bridges ordinarily need repainting every three or four years.

TABLE LXVIII.

AVERAGE SURFACE COVERED PER GALLON OF PAINT.

Paint.	Volume of Oil.	Pounds of Pigment.	Volume and Weight of Paint.	Square Feet.	
				1 Coat.	2 Coats.
			Gals. Lbs.		
Iron oxide (powdered)....	1 gal.	8.00	1.2 = 16.00	600	350
Iron oxide (ground in oil).	1 "	24.75	2.6 = 32.75	630	375
Red lead (powdered)......	1 "	22.40	1.4 = 30.40	630	375
White lead (ground in oil).	1 "	25.00	1.7 = 33.00	500	300
Graphite (ground in oil)...	1 "	12.50	2.0 = 20.50	630	350
Black asphalt.......	1 " (turp.)	17.50	4.0 = 30.00	515	310
Linseed oil (no pigment).	1 "	....	....	875	......

Light structural work will average about 250 square feet, and heavy structural work about 150 square feet of surface per net ton of metal, while No. 20 corrugated steel has 2,400 square feet of surface.

It is the common practice to estimate ½ gallon of paint for the first coat and ⅜ gallon for the second coat per ton of structural steel, for average conditions.

The price of paint materials in small quantities in Chicago are (1908) about as follows: Linseed oil, 50 to 60 cents per gal.; iron oxide, 1 to 2 cents per lb.; red lead, 7 to 8 cents per lb.; white lead, 6 to 7 cents per lb.; graphite, 6 to 10 cents per lb.

A good painter should paint 1,200 to 1,500 square feet of plate surface or corrugated steel or 300 to 500 square feet of structural steel work in a day of 8 hours; the amount covered depending upon the amount of staging and the paint. A thick red lead paint mixed with 30 lbs. of lead to the gallon of oil will take fully twice as long to apply as a graphite paint or linseed oil.

The amount of paint and the cost of labor in painting highway bridges, as computed by the Youngstown Bridge Co., are given in Table LXIX. A common rule has been to ship one gallon of paint for each 10 feet of bridge for each field coat.

For additional information on paints and painting, see "The Design of Steel Mill Buildings," Chapter XXVII.

TABLE LXIX.

AMOUNT OF PAINT IN GALLONS AND COST OF LABOR IN PAINTING HIGHWAY BRIDGES (YOUNGSTOWN BRIDGE CO.).

Span Length, Feet.	Highway.		Oxide Iron, Gals. per Coat.		Red Lead, Gals. per Coat.		White Lead, Gals. per Coat.		Graphite, Gals. per Coat		Asphalt, Gals. per Coat.		Carbon Primer, Gals. per Coat		Cost 100 Pounds Paint	Labor, One Coat.
	Weight, Pounds.	Area, Square Feet.	1st	2d	1st	2d	1st	2d	1st	2d	1st	2d	1st	2d		
20	1,800	400	1	3/4	[illegible]	1/2	1	3/4	1	3/4	1 1/2	1	1/2	1/2	.025	2.50
40	5,200	800	1 1/2	1 1/4	[illegible]	1	1 1/2	1 1/4	1 1/2	1 1/4	3	1 1/2	3/4	1/2	.015	4.25
60	10,200	1,400	3	2	[illegible]	1 1/2	3	2	3	2	5	3	1 1/2	1	.015	8.80
80	16,800	2,200	4 1/2	3	[illegible]	2 1/4	4 1/2	3	4 1/2	3	7	4 1/2	2 1/4	1 1/2	.013	10.20
100	25,000	3,000	6	4 1/2	[illegible]	3	6	4 1/2	6	4 1/2	10	6	3	2	.012	14.50
120	34,800	3,800	7 1/2	5 1/2	[illegible]	4	7 1/2	5 1/2	7 1/2	5 1/2	13	7 1/2	3 3/4	2 1/2	.012	18.80
140	46,200	5,400	11	8	[illegible]	5 1/2	11	8	11	8	18	11	5 1/2	3 1/2	.012	27.20
160	59,200	8,000	16	12	[illegible]	8	16	12	16	12	27	16	8	5 1/2	.013	41.00
180	73,800	10,000	20	14	[illegible]	10	20	14	20	14	33	20	10	7	.013	47.50
200	90,000	12,000	24	17	[illegible]	12	24	17	24	17	40	24	12	8	.013	57.80
220	107,800	15,000	30	21	[illegible]	15	30	21	30	21	50	30	15	10	.014	71.50
240	127,200	18,000	36	26	[illegible]	18	36	26	36	26	60	36	18	12	.014	89.40
260	148,200	21,000	42	30	[illegible]	21	42	30	42	30	70	42	21	14	.014	102.00
280	170,800	24,000	48	35	[illegible]	24	48	35	48	35	80	49	24	16	.014	119.00
300	195,000	28,000	56	40	[illegible]	28	58	40	56	40	90	56	28	19	.014	136.00

COST OF MASONRY ABUTMENTS AND PIERS.—The cost of masonry abutments and piers varies between wide limits, depending upon the cost of stone, cost of quarrying, cost of dressing, cost of laying, cost of mortar, cost of superintendence, cost of tools, cost of maintenance and depreciation of plant. Space will not permit a discussion of all the above items.

Cost of Stone.—The price of stone is usually quoted f. o. b. at the quarry, and varies with the stone and location.

Cost of Quarrying.—After the quarry has been opened in limestone, two-man stone for rubble wall can usually be quarried for from 1/4 to 1/2 the cost of the daily wages of a quarry laborer per cu. yd. Stones ranging from 1/2 to 1 cu. yd., that have to be blasted, will cost per cu. yd. from 1/2 to 2 times the cost of the daily wages of one man. Dimension stones that have to be wedged out will cost twice as much as the large stones that can be blasted. This estimate is high for sandstone and low for granite.

Cost of Dressing.—Rubble is roughly scabbled when it is laid and there is no special charge for dressing. Dimension stones, if dressed

to lay with quarry finish and fairly close joints, will cost from $1.00 to $3.00 per cu. yd. Bush-hammering costs about 25 cents per sq. ft.

Cost of Laying.—One mason and a helper can lay from 4 to 5 cu. yds. of small rubble in a day of 8 hours. If a derrick is necessary and some dressing required, one mason and a helper will lay only from 2 to 3 cu. yds. of heavy rubble or 1½ to 2 cu. yds. of dimension stone in a day of 8 hours.

Cost of Mortar.—The amount of mortar required varies with the specifications and the stone used. Rubble masonry is from 20 to 35 per cent mortar. Dimension stone masonry is from 10 to 15 per cent mortar. Knowing the cost of cement and sand, the cost of the mortar can be estimated.

Miscellaneous Costs.—The cost of superintendence, tools, maintenance and depreciation of plant, etc., can only be estimated on the particular work. These costs may vary from 5 to 20 per cent of the cost above.

COST OF REINFORCED CONCRETE ABUTMENTS AND PIERS.—The cost of reinforced concrete may be divided into cost of cement, cost of sand, cost of broken stone, cost of forms, cost of laying, cost of reinforcement, cost of superintending, etc.

TABLE LXX.*

INGREDIENTS IN 1 CUBIC YARD OF CONCRETE: SAND VOIDS, 40 PER CENT; STONE VOIDS, 45 PER CENT; PORTLAND CEMENT, BARREL YIELDING 3.65 CU. FT. PASTE. BARREL SPECIFIED TO BE 3.8 CU. FT.

Proportions by Volume.	1:2:4	1:2:5	1:2:6	1:2½:5
Barrels Cement per cubic yard Concrete......	1.46	1.30	1.18	1.13
Cubic yards Sand " " " "	0.41	0.36	0.33	0.40
Cubic yards Stone " " " "	0.82	0.90	1.00	0.80
Proportions by Volume.	**1:2½:6**	**1:3:4**	**1:3:5**	**1:3:6**
Barrels Cement per cubic yard Concrete......	1.00	1.25	1.13	1.05
Cubic yards Sand " " " "	0.35	0.53	0.48	0.44
Cubic yards Stone " " " "	0.84	0.71	0.80	0.88
Proportions by Volume.	**1:3:7**	**1:4:7**	**1:4:8**	**1:4:9**
Barrels Cement per cubic yard Concrete......	0.96	0.82	0.77	0.73
Cubic yards Sand " " " "	0.40	0.46	0.43	0.41
Cubic yards Stone " " " "	0.93	0.80	0.80	0.92

Cement is to be measured packed in the barrel.

* From Gillette's "Cost Data," p. 255.

Ingredients in Concrete.—With sand voids 40 per cent, stone voids 45 per cent, and a Portland cement barrel holding 3.8 cu. ft., Gillette gives relative quantities of cement, sand and broken stone as in Table LXX. The cement is measured packed in the barrel. The quantities for other percentages of voids differ considerably. If the cement is measured loose the quantities of sand and stone are practically the same, but the amount of cement is reduced from 10 to 15 per cent.

Cost of Materials.—The cost of concrete materials varies with the location and must be obtained before preparing an estimate of cost. Portland cement (1908) costs about $1.25 per barrel f. o. b. works; sand can usually be obtained delivered on wagons at $0.75 to $1.00 per cu. yd.; broken stone can usually be obtained at $1.00 to $1.50 per cu. yd. f. o. b. crusher.

Cost of Forms.—The cost of the timber for forms should be obtained locally. Lumber can be used several times, commonly from 3 to 5 times where forms are torn down each time, and almost indefinitely where forms can be used without change; so that the first cost of lumber should be distributed over all the concrete laid with the forms. The amount of lumber per cubic yard varies with the type of wall, being very much greater for reinforced than for plain walls. A carpenter and a helper should be able to place one M ft. B. M. in two days of 8 hours each, making the cost from $6 to $8 per M ft. B. M.

Cost of Mixing and Placing.—With men at $1.50 per day and a foreman at $3.00 per day, concrete should be mixed and rammed into place for from $0.75 to $1.25 per cu. yd. If the reinforcement is troublesome or the wall narrow the cost may be higher. This does not include facing which will cost more. If machine mixers are used the cost is less on large work.

The following is an approximate estimate for concrete abutments containing 300 cu. yds., the concrete being mixed by hand:

1.05 bbls. Portland cement @ $2.50	$2.625
0.40 cu. yd. sand @ $1.00	0.40
0.90 cu. yd. stone @ $1.25	1.125
Labor mixing and placing concrete	1.00
Lumber for forms @ $20.00 per M	.80
Labor on forms @ $8.00 per M	.28
Total per cu. yd	$6.230

TABLE LXXI.

COST OF CONCRETE ABUTMENTS FOR 18 STEEL HIGHWAY BRIDGES ON THE ILLINOIS AND MISSISSIPPI CANAL.

	Excavation.		Back-filling.		Natural Cement Concrete.		Portland Cement Concrete.		Slope Paving.		Piles Driven.	
	Cubic Yards.	Cents.	Cubic Yards.	Cents.	Cubic Yards.	Dollars	Cubic Yards.	Dollars	Cubic Yards.	Dollars	Linear Feet.	Cents.
Highest cost.	117.0	36	200	26	252	6.54	248	8.86	340	2.39	768	63
Lowest cost..	574.3	21	979	18¼	947	4.98	248	6.96	340	1.82	768	42
Average cost	550.0	26	600	20	200	6.09	248	8.00	320	2.10	768	50

The highest, lowest and average costs for building the concrete abutments on the Illinois and Mississippi Canal are given in Table LXXI.

ESTIMATED COST OF A RIVETED TRUSS HIGHWAY BRIDGE.—A detailed estimate will be made of a 111′ 6″ riveted Pratt steel highway bridge over the Illinois and Mississippi Canal, the detail shop plans of which are given in Figs. 162 and 163.

Cost of Material.—The cost of the steel will be estimated at the mill at Pittsburg, Pa.

Bridge, Exclusive of Joists and Fence.—The bridge is composed of beams, angles, bars, plates and pin rounds as given in the following table:

Shape.	Weight, Pounds.	Per Cent of Total Weight.	Cost Per Pound, Cents.	Percentage of Pound Cost, Cents.
Beams and Channels......	22,278	39	1.70	0.663
Angles 3″ and under......	15,779	27	1.70	0.459
Angles over 3″............	1,021	2	1.60	0.032
Bars..........................	11,585	20	1.60	0.320
Plates........................	6,222	11	1.60	0.176
Pin Rounds..................	506	1	2.00	0.020
Total....................	57,393	100		1.670

The average cost of the steel at the mill............	1.670	cts. per lb.
Waste in fabrication, 4 per cent..................	0.067	" "
Paint material	0.010	" "
Freight, Pittsburg to Chicago....................	0.165	" "
Average cost of the steel at the shop........	1.912	" "

Joists.—The joists, end struts and hub guards (fence) will take the rate of 1.70 cts. per lb. at the mill.

The average cost of the steel at the mill	1.700	cts. per lb.
Waste in fabrication, 2 per cent	0.034	" "
Paint material	0.010	" "
Freight, Pittsburg to Chicago	0.165	" "
Average cost of the steel at the shop	1.909	" "

Shop Cost of the Steel in the Bridge, Exclusive of Fence, Joists, etc.—

Average cost of steel at the shop	1.912	cts. per lb.
Shop cost, including drafting	0.750	" "
Total shop cost	2.662	" "
Freight, shop to railroad station near site	0.100	" "
Total cost at railroad station	2.762	" "

Shop Costs of Joists, Fence and End Struts.—

Average cost of the steel at the shop	1.909	cts. per lb.
Shop cost, including drafting	0.250	" "
Total shop cost	2.159	" "
Freight, shop to railroad station near site	0.100	" "
Total cost at railroad station	2.259	" "

Erection.—

Hauling 43 tons 4 miles, @ 25 cts. per ton mile for hauling and 50 cts. per ton for loading and unloading	$ 64.50
Falsework.—Twenty piles 35 ft. long @ 15 cts. per ft.	105.00
Driving 525 lin. ft. piling @ 25 cts.	131.25
Timber, 6,000 ft. B. M.—½ price—@ $12.00	72.00
Placing timber, 6,000 ft. B. M. @ $8.00	48.00
Labor erecting and bolting the steel, 30 days, labor @ $4.00	120.00
Transportation of men and tools	60.00
Driving 1,500 field rivets @ 10 cts.	150.00
Labor, painting bridge 2 coats, 10 days @ $4.00	40.00
Labor, erecting floor lumber, 12,000 ft. B. M. @ $4.00	48.00
Total cost of erection	$838.75

Summary of Cost of Superstructure.—

Steel, 57,393 lbs. @ 2.762 cts. per lb.	$1,685.10
Joists, fence, etc., 26,713 lbs. @ 2.259 cts.	603.45

Lumber—yellow pine, 5,715 ft. B. M. @ $25.00	$ 142.87
Lumber—oak plank, 6,540 ft. B. M. @ $32.00	209.28
Paint, 20 gallons @ $1.25 per gallon	25.00
Bolts for the floor, 400 lbs. @ 3 cts	12.00
Spikes for the floor, 400 lbs. @ 3 cts	12.00
Cost of erection	838.75
Total cost of the superstructure	$3,528.45

Contract Price.—

Steel in place, 84,106 lbs. @ 4 cts. per lb	$3,364.24
Yellow pine in place, 5,715 ft. B. M. @ $36.00	205.74
Oak timber in place, 6,540 ft. B. M. @ $46.00	300.84
Total contract price	$3,870.82
Profit	$ 342.37

CHAPTER XX.

General Principles of Design of Highway Bridges.

The Economic Bridge.—The most economic bridge for any crossing is that one which in the long run will give the best service and cost the least money. As a measure of the cost we may take the amount of money which it will be necessary to capitalize in order that the interest will pay for the maintenance and repairs, the interest on the first cost, and provide a sinking fund for depreciation so that the bridge may be replaced when it is no longer fit for the service demanded. The possible life of any particular bridge is difficult to estimate. Steel bridges, if properly constructed, should last from 25 to 40 years; combination bridges from 12 to 15 years; timber bridges from 10 to 15 years (covered Howe truss bridges have given good service for 40 to 50 years); while masonry and reinforced concrete bridges are ordinarily considered as permanent structures. Timber floors must be replaced in from 3 to 5 years, and steel bridges should be painted every 3 to 4 years. Many steel bridges have proved to be too light for electric railways and other service and have had to be replaced before the bridge was worn out.

From the foregoing discussion it will be seen that the first cost of a bridge is not a safe criterion upon which to base economy. Much money is wasted by putting in cheap, flimsy bridges which are short-lived, unsatisfactory and are continually in need of repairs. The following discussion will be of assistance in determining the most economic number of spans for a given crossing.

Experience and theoretical investigations have shown that the most economical number of spans for a given crossing occurs where the cost of the superstructure is equal to the cost of the substructure. Where the bridge is built across a shallow stream in which the piers can be placed at any point with equal ease, this will give all spans of equal length. This case does not often occur in practice. With a deep channel in which the piers are relatively expensive, the channel spans are made longer than the shore spans. Where there is a single span there will be two abutments; where there are two spans there will be two abut-

ments and one pier; where there are three spans there will be two abutments and two piers, etc. It will be seen that where two or three spans are used in a crossing the spans should be shorter than where a single span is used.

The author has solved this problem in practice by trial, using tables of weights of bridges and estimated costs of substructure. It should be remembered in this connection that local conditions often determine the number and lengths of the spans so that the criterion is not always applicable; for example, the bottoms of the streams flowing into the Missouri River wash so that it is very risky to put a pier in the stream; the main span must then entirely clear the channel.

As an example it is required to determine the most economic number of spans for a Pratt pin-connected highway bridge on masonry abutments and piers, to cross a shallow stream 300 feet wide, requiring abutments and piers 30 feet high. The bridge is to be designed for a live load of 1,200 lbs. per lineal foot of bridge, the roadway is to be 20 ft. in the clear. It will be assumed that masonry costs $10.00 per cu. yd. in place, lumber $30.00 per M in place, and the steel in the bridge costs 4 cts. per lb.

An estimate of the cost of three 100-ft. spans, two 150-ft. spans, and one 300-ft. span will be calculated. From Table XLVI the abutments for a 100-ft. span will contain 206 cu. yds., while the wings will be estimated at 94 cu. yds. per abutment, or a total of 300 cu. yds. per abutment. The abutments for the 300-ft. span will contain 330 cu. yds., and the abutments for the 150-ft. span may be directly interpolated. From Table L the piers for 100-ft. spans will contain 136 cu. yds., while the piers for the 150-ft. spans will contain 157 cu yds.

The steel in a 100-ft. span with 6 panels, from Fig. 40, will weigh: trusses = 13,000 lbs.; Fig. 35, floorbeams = 1.300 × 7 = 9,100 lbs.; Fig. 36, laterals = 250 × 20 = 5,000 lbs. The total weight of steel in the bridge exclusive of joists = 27,100 lbs. The weight of the joists from Table VIII = 15,350 lbs. The total weight of the steel in one span = 42,450 lbs.

The steel in a 150-ft. truss with 9 panels, from Fig. 40, will weigh: trusses = 32,000 lbs.; Fig. 35, floorbeams = 1,300 × 9 = 11,700 lbs.; Fig. 36, laterals = 550 × 20 = 11,000 lbs. The total weight of the steel in the bridge exclusive of joists = 54,700 lbs. The weight of the joists from Table VIII = 23,025 lbs. The total weight of the steel in one span = 77,725 lbs.

The steel in a 300-ft. span Petit truss bridge with 14 panels is:

trusses, from Fig. 41 = 130,000 lbs.; from Fig. 35, floorbeams = 1,600 $\times$ 15 = 24,000 lbs.; from Fig. 36, laterals = 1,400 $\times$ 20 = 28,000 lbs. The total weight of the steel in the span exclusive of the joists = 182,000 lbs. The weight of the joists from Table VIII = 46,500 lbs. The total weight of the steel in one span = 228,500 lbs.

With 3 inch plank in the floor, three 4″ $\times$ 6″ spiking pieces, two 4″ $\times$ 6″ felloe guards and a wooden fence there will be 80 ft. B. M. per lineal foot of bridge, or 24,000 ft. B. M. in the bridge.

Cost of three 100-ft. spans:		
Two abutments, 600 cu. yds. @ \$10.00........	\$6,000.00	
Two piers, 272 cu. yds. @ \$10.00.............	2,720.00	
Total cost of substructure.............		\$ 8,720.00
Cost of steel, 3 × 42,450 lbs. @ 4 cts.........	5,094.00	
Cost of lumber, 24,000 ft. B. M. @ \$30.00.....	720.00	
Total cost of superstructure...........		5,814.00
Total cost of bridge..................		\$14,534.00
Cost of two 150-ft. spans:		
Two abutments, 620 cu. yds. @ \$10.00........	\$6,200.00	
One pier, 157 cu. yds. @ \$10.00..............	1,570.00	
Total cost of substructure.............		\$ 7,770.00
Cost of steel, 2 × 77,725 lbs. @ 4 cts.........	6,218.00	
Cost of lumber, 24,000 ft. B. M. @ \$30.00.....	720.00	
Total cost of superstructure...........		6,938.00
Total cost of the bridge...............		\$14,708.00
Cost of one 300-ft. span:		
Two abutments, 660 cu. yds. @ \$10.00........	\$6,600.00	
Cost of steel, 228,500 lbs. @ 4 cts............	9,140.00	
Cost of lumber, 24,000 ft. B. M. @ \$30.00.....	720.00	
Total cost of bridge..................		\$16,360.00

It will be seen that for the given conditions the most economical crossing is with the three 100 ft. spans. With masonry at \$15.00 per cu. yd. in place, the costs will be: three 100-ft. spans, \$18,994.00; two 150-ft. spans, \$18,593.00; one 300-ft. span, \$19,760.00.

Riveted vs. Pin-connected Truss Bridges.—Riveted bridges should be used for all low truss bridges and preferably for high truss spans up to spans of, say, 150 feet. Spans longer than 150 feet should be

pin-connected and all high truss spans may be pin-connected. Pin-connected high truss bridges should never be built with less than 5 panels, for the reason that it is practically impossible to counter-brace Pratt trusses with less than this number of panels. Riveted low trusses cost practically the same as pin-connected low truss bridges, while riveted high truss bridges usually cost slightly more than pin-connected bridges having the same capacity and dimensions. Riveted bridges are usually more rigid than pin-connected bridges.

Preliminary Plans.—The preliminary plans of bridges may consist (1) of a stress diagram showing the stresses, dimensions and sizes of the principal members of the bridge, and also standard specifications as given in Appendix I; (2) of detail plans which show the make-up of all the members together with the maximum and minimum spacing of the rivets, thickness and sizes of plates, lacing bars, etc., and also standard specifications; and (3) of completely detailed shop plans and specifications. When properly carried out all of the methods will give satisfactory results. Ordinarily the customer cannot understand the details of the bridge from a study of the stress diagram and finds the second and third methods much more satisfactory. It is seldom profitable to prepare shop plans until after the order for the bridge has been placed in the shop, and requisitions have been made for the material. On the whole, the method of preparing the preliminary plans, as described in (2), is the most satisfactory. This makes it possible to specify exactly the details of the sections and at the same time permits the bridge shop to follow its own methods wherever possible. The shop practice in different shops differs so much that it is ordinarily cheaper for the bridge company to prepare its own shop plans than to follow shop plans that have been prepared by engineers that are not familiar with the particular shop.

Bridge Lettings.—The contracts for building highway bridges are ordinarily let by county commissioners, county surveyors or other county officers. In a few states contracts for building bridges are let by state or highway engineers. The common method of awarding contracts for highway bridges has been about as follows: Three or four weeks before the date set for the bridge letting the county clerk or other officer advertises that bids will be received up to a certain hour for building a certain bridge or bridges, and that the bids will then be publicly opened and the contract awarded. The main dimensions and the capacity only are ordinarily specified and the bidders are asked to

submit their own plans and specifications. When the various bids and plans are received the commissioners are entirely at a loss as to what is the best thing to do, and the result is that either the contract is given to the lowest bidder on a very poor plan or is given to a favorite bidder on a plan that results in a worse bridge. This loose method of contracting for bridges makes it practically impossible for even honest officials to procure a satisfactory structure and opens up the way for dishonest officials and contractors to arrange a deal whereby the public comes out second best. It also makes it possible for the contractors to "pool" so that the bridge contract will go to a member of the pool at an agreed price. The county surveyor or local engineer is ordinarily not much better posted on the merits of the bids and plans than the commissioners and their participation in the letting does not ordinarily improve matters.

The practice of "bridge pooling" is disreputable and has worked to the disadvantage of both the public and of reputable bridge companies. It has made it possible for "fake bridge companies" to exist and also for crooked public officials to receive part of the profits of the transaction. It has uniformly resulted in high prices and poor bridges.

Before advertising for bids the matter of the design of the bridge or bridges should be placed in the hands of a competent consulting bridge engineer. Detail plans and specifications should be prepared and an estimate of the probable cost submitted to the officials. All bids should then be received on the official plans and specifications. If the bids are too high they should all be rejected and the work readvertised on the same or on revised plans. The bridge contractor takes a considerable risk and is entitled to a good legitimate profit, and the engineer should add 15 to 20 per cent for profit to his estimated cost. No work should be done at the shop until after the shop plans have been checked and approved by the consulting engineer. The shop, field and final inspection should be in the hands of the consulting engineer. This method will meet the approval of all legitimate bridge companies, and will result in better bridges at a cost less than that of the present miserable structures.

Contract.—After the contract has been awarded the contract should be drawn up and signed, and an indemnity bond should be furnished by a good surety company. Sample contract and bond forms used by the author follow:

BRIDGE CONTRACT.

Agreement made this .... day of ...., 19.., by and between ...., a corporation of the State of ...., party of the first part, and ...., party of the second part.

Witnesseth, that for the consideration and upon the terms and conditions hereinafter provided, the party of the first part agrees to furnish all material and labor therefor, and construct and erect in a good and workmanlike manner .... over the .... called .... at a point where the .... crosses said .... in the .... of ...., County of ...., and State of ...., according to the attached plans and specifications which are made a part of this contract: The bridge is to have ...... spans; extreme length of each span, ...; space between the face of abutments, ...; roadway, ... feet clear; sidewalk, ... feet clear. The abutments to be ....; the piers to be ....

It is further agreed that the said first party shall save and hold said second party free and harmless from any and all claims for damages to life, limb or property occasioned or caused by said first party's employees; and from all claims for materials and labor furnished on this contract.

The party of the first part agrees to complete the work herein contracted for, and to have the said bridge open and ready for travel on or before the .... day of ...., 19...

The party of the second part agrees to pay the party of the first part for said bridge the sum of .... in cash, as follows: ...... per cent upon the delivery by the party of the first part of the steel and other material on the bridge site for said bridge, ... per cent additional upon the completion of the erection of the different parts of the bridge, and the remaining ...... per cent upon the completion and acceptance of the bridge by the consulting engineer of the second party. Estimates of material delivered and work done shall be made by the consulting engineer not later than the 5th day of the month for all material delivered or work done during the preceding month, and the payment will be made on or before the 15th of the month for the material delivered and work done the preceding month.

It is further stipulated and agreed that the party of the first part shall furnish to the party of the second part an indemnity bond in an approved surety company in the sum of .... dollars.

It is further stipulated and agreed that for the failure of the first party to complete the bridge as stipulated the said first party shall forfeit .... dollars for each working day until the bridge is completed. This sum to be considered as liquidated damages, and not as a penalty.

(It is further stipulated and agreed that the party of the first part shall not be held responsible for delays in delivery of material from the rolling mills, nor delays in transportation, nor for delays occasioned by strikes, fires, floods, storms, or other circumstances beyond its control, but may be granted an extension of time as may be determined by the consulting engineer of the second party.)

In Witness Whereof, the said parties to this agreement hereunto set their hands and seals as of the day and year first above written.

........................First Party.
......................Second Party.

BOND.

Know all Men by These Presents, that ...., as party of the first part, and the ...., a corporation organized and existing under the laws of the State of .... as Surety, are held and firmly bound unto the ...., party of the second part, its successors and assigns, in the sum of .... dollars, lawful money of the United States, to the payment of which sum well and truly to be made the said first party and the said Surety do hereby bind themselves, their heirs, executors, administrators, successors and assigns, jointly and severally, firmly by these presents.

Signed, Sealed, Dated and Delivered, this .... day of ...., 19...

WHEREAS, the said party of the first part has entered into a certain contract with the ...., dated ...., 19.., for ....

....

....

which contract is hereto attached and made a part hereof, and for a fuller description thereof reference is made to said contract:

NOW, THEREFORE, THE CONDITION OF THIS OBLIGATION IS SUCH, that if the said party of the first part shall and does pay as they become due, all just claims for all work and labor performed and all skill and material furnished in the execution of such contract, and, also, shall save the party of the second part named in this bond harmless from any cost, charge and expense that may accrue on account of the doing of the work specified in such contract according to the terms thereof and the contract price therein, and shall comply with all the requirements of the law, then this obligation shall be void; otherwise to remain in full force and effect.

IN TESTIMONY WHEREOF, We have hereunto set our hands and seals, this .... day of ...., 19...

Signed, sealed and delivered
in the presence of

....

....Seal.

............................Seal.

STATE OF.................... }
COUNTY OF.................. } SS.

On this ...... day of ...., 19.., before me, a Notary Public in and for the County and State aforesaid, personally came ....

....

to me known, who, being by me duly sworn, did depose and say that he resided in ....; that he is the ....

of the ....

the corporation described in and which executed the above bond as party of the first part; that he knew the seal of said corporation; that the seal affixed to said instrument was such corporate seal; that it was so affixed by order of the board of directors of said corporation and that he signed his name thereto by like order.

....

Notary Public.

STATE OF.................... } ss.
COUNTY OF.................. }

On this ...... day of ...., 19.., before me, a Notary Public in and for the County and State aforesaid, personally came .... to me known, who, being by me duly sworn, did depose and say that he resided in ....; that he is the .... of the ...., the corporation described in and which executed the above bond as Surety; that he knew the seal of said corporation; that the seal affixed to said instrument was such corporate seal; that it was so affixed by order of the board of directors of said corporation and that he signed his name thereto by like order.

....
Notary Public.

PART III.

A PROBLEM IN HIGHWAY BRIDGE DETAILS.

Introduction.—The best way for the student to become familiar with bridge details is for him to make a critical investigation of an existing structure. The problem given in this discussion has been used by the author in his classes preliminary to a course in bridge design. The problem includes the calculation of the weight, the cost and the efficiencies of the members of a 160-ft. pin-connected Pratt highway bridge. The shop plans of the bridge are shown in Fig. 285. This bridge has been selected for the reason that, while it is designed for the most part on correct lines, it has at the same time most of the defects that are liable to occur in a highway bridge. These errors are criticised and the proper design is suggested.

(Title page of Problem.)

INVESTIGATION OF A 160′ 0″ × 16′ 0″ HIGHWAY BRIDGE OVER THE LICKING RIVER, NEAR ORRICK, MO.

BUILT BY

A. B. C. BRIDGE COMPANY,

CHICAGO, ILLINOIS.

ORDER 7,530

1906.

Signed: JOHN J. JONES.

UNIVERSITY OF COLORADO.

1908.

Location and General Description.—The bridge is over Licking River, four miles from Orrick, Missouri, and was erected by the A. B. C. Bridge Company in 1906.

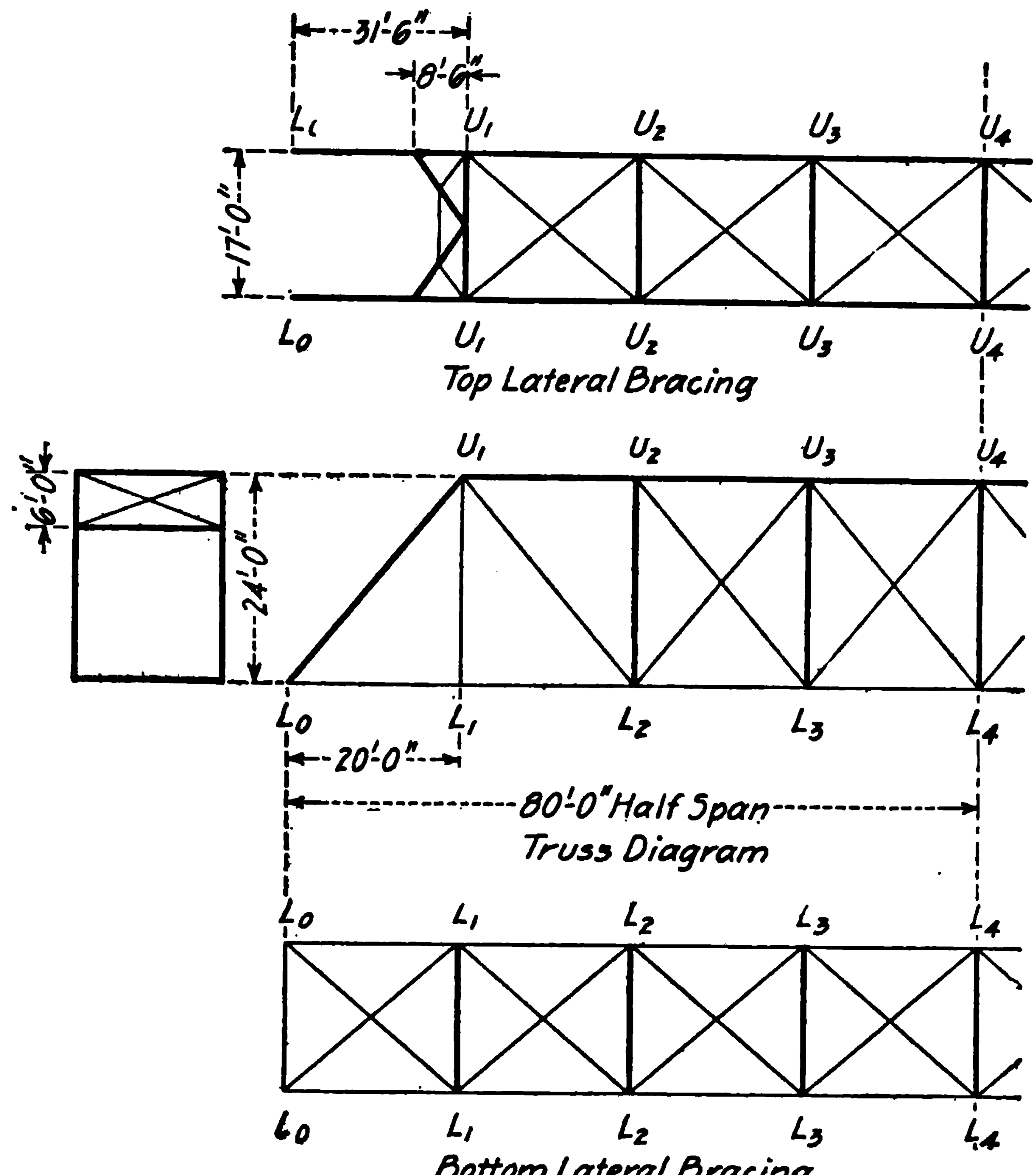


FIG. 284.

The bridge is 160′ 0″ center to center of end pins and has a clear roadway of 16′ 0″, as shown in Fig. 284.

CHAPTER XXI.

Calculation of Weight and Cost of a 160-ft. Span Pratt Highway Bridge.

It is assumed that the shipping weight of the bridge as it was shipped from the bridge shop is required. The weights of the members in the order, end-posts, top chords, etc., are calculated in detail. The "main members" are those that are given on the stress sheet and are either members in which stresses occur or which are specified by the designing engineer; while the "details" are plates, angles, rivets, etc., which are necessary to develop the strength of the "Main members." The reference number is given in column 1; the number of pieces of the same kind is given in column 2; the shape of the section is given in column 3; the size of the section is given in column 4; the length of the plate, channel, etc., is given in column 5; the weight per foot is given in column 6—the weight of rivets and similar details is given as "lbs. per 100"; the weight of "main members" and "details" is given in column 7; the "details" in per cent of "main members" is given in column 8; while the total weight is given in column 9. The weights of rivet heads, only, are included in the weight of the structure, it being assumed that the remainder of the rivet fills up the hole punched in the member. The weights of rivet heads were taken from Cambria Steel, and are larger than the weights given by Carnegie. The chord pins and nuts, and pedestals are considered as "details." A summary of the weight of the metal in the bridge is given in Table LXXIII. The weight of the lumber is given in Table LXXIV. In Table LXXV the weights of the different parts of the bridge and the per cent of "details" are compared with two other highway bridges.

For a discussion of the per cent of details in highway bridges of different types, see Chapter XIX.

References are made to "Cambria" (Cambria Steel, 1907 Edition) and to "Carnegie" (Carnegie Steel Handbook, 1903 Edition).

TABLE LXXII.

Estimate of Weight

Steel Pratt Through Highway Bridge, Span 160'0"; Roadway 16'0"; Depth 24'-0".

1	2	3	4	5		6	7		8	9
Ref. No.	No. of Pieces	Shape	Section	Length		Wt. per Foot	Weight		Details Per Cent of Main Members	Total Weight
				Ft.	In.		Main Members	Details		
1	4	End Posts, L_0U_1, each thus:-								
	2	[s	10"@15#	31	6	15.00	945			
	1	Cover Pl.	12"×1/4"	31	8 1/2	10.20	323			
	1	Batten Pl.	12"×1/4"	1	0	10.20		10		
	1	" "	12"×1/4"	0	10	10.20		8		
	2	Hinge Pls.	8"×1/4"	0	11 1/2	6.80		14		
	2	Pin Pls.	8"×5/16"	0	8 1/2	8.50		12		
	2	Pin Pls.	8"×7/16"	1	4	11.90		32		
	2	" "	8"×3/8"	1	6 11/16	10.20		31		
	2	" "	8"×1/4"	0	11 7/16	6.80		14		
	56	Lacing Bars	2"×5/16"	1	2	2.12		138		
	628	Rivet Heads	3/4"φ	per	100	16.12		101		
							1268	360	28.5	
			Total Weight of 4 End Posts = 1628 × 4 =							6512
2	2	Top Chords, U_1U_2---U_1', each thus:-								
	2	Cover Pls.	12"×1/4"	40	3	10.20	821			
	1	" Pl.	12"×1/4"	40	1	10.20	409			
	4	[s	10"@15#	39	0	15.00	2340			
	2	[s	10"@15#	42	7	15.00	1278			
	12	Batten Pls.	12"×1/4"	1	0	10.20		122		
	4	Hinge Pls.	8"×1/4"	1	2	6.80		32		
	4	Pin Pls.	8"×3/8"	0	11	10.20		37		
	4	" "	8"×3/8"	1	6 1/2	10.20		63		
	6	" "	8"×1/4"	1	4	6.80		54		
	4	Splice Pls.	8"×5/16"	2	5	8.50		82		
	8	[Clips	7"[@9 3/4#	0	7	9.75		47		
	4	[Clips	6"[@8#	0	6	8.00		16		
	198	Lacing Bars	2"×5/16"	1	2	2.12		489		
	2270	Rivet Heads	3/4"φ	per	100	16.12		366		
							4848	1308	27.0	
			Total Weight of 2 Top Chords = 6156 × 2 =							12312
3	8	Lower Chords, L_0L_1, L_1L_2, each thus:-								
	2	Eye Bars	2 1/2"×7/8"	20	0	7.44	298			
	4	Eyes	2 1/2"×7/8"	1	1 1/2	7.44		34		
							298	34	11.4	
			Total Weight of 8 Lower Chords = 332 × 8 =							2656
							Carried Forward			21480

1	2	3	4	5		6	7		8	9
Ref. No.	No. of Pieces	Shape	Section	Length		Wt. per Foot	Weight		Details Per Cent of Main Members	Total Weight
				Ft.	In.		Main Members	Details		
							Brought Forward			21480
3	4	Lower Chords, L_2L_3, each thus:-								
	2	Eye Bars	$3\frac{1}{2}'' \times \frac{15}{16}''$	20	0	11.16	446			
	4	Eyes	$3\frac{1}{2}'' \times \frac{15}{16}''$	1	6	11.16		67		
							446	67	15.0	
				Total Weight of 4 Lower Chords = 513 × 4 =						2052
	4	Lower Chords, L_3L_4, each thus:-								
	4	Eye Bars	$3'' \times \frac{11}{16}''$	20	0	7.02	562			
	8	Eyes	$3'' \times \frac{11}{16}''$	1	6	7.02		84		
							562	84	15.0	
				Total Weight of 4 Lower Chords = 646 × 4 =						2584
4	4	Intermediate Posts, U_2L_2, each thus:-								
	2	[s	$7''$ @ $9\frac{3}{4}$#	25	3	9.75	492			
	4	Batten Pls.	$10'' \times \frac{1}{4}''$	0	10	8.50		29		
	2	Pin Pls.	$7'' \times \frac{1}{2}''$	1	6	11.90		36		
	2	" "	$5'' \times \frac{3}{8}''$	1	$1\frac{1}{2}$	6.38		14		
	4	Ls	$2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{4}''$	1	0	4.00		16		
	100	Lacing Bars	$1\frac{3}{4}'' \times \frac{1}{4}''$	0	$11\frac{1}{2}$	1.49		148		
	264	Rivet Heads	$\frac{5}{8}''\phi$	per	100	9.95		26		
	112	" "	$\frac{3}{4}''\phi$	"	100	16.12		18		
							492	287	58.3	
				Total Weight of 4 Intermediate Posts = 779 × 4 =						3116
	4	Intermediate Posts, U_3L_3, each thus:—								
	2	[s	$7''$ @ $9\frac{3}{4}$#	25	3	9.75	492			
	4	Batten Pls.	$10'' \times \frac{1}{4}''$	0	10	8.50		29		
	2	Pin Pls.	$5'' \times \frac{3}{8}''$	0	11	6.38		12		
	2	" "	$7'' \times \frac{3}{8}''$	1	$3\frac{1}{2}$	8.93		24		
	4	Ls	$2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{4}''$	1	0	4.00		16		
	100	Lacing Bars	$1\frac{3}{4}'' \times \frac{1}{4}''$	0	$11\frac{1}{2}$	1.49		148		
	264	Rivet Heads	$\frac{5}{8}''\phi$	per	100	9.95		26		
	92	" "	$\frac{3}{4}''\phi$	"	100	16.12		15		
							492	270	54.9	
				Total Weight of 4 Intermediate Posts = 762 × 4 =						3048
	2	Intermediate Posts, U_4L_4, each thus:-								
	2	[s	$6''$ @ 8#	25	3	8.00	404			
	4	Batten Pls.	$9\frac{1}{2}'' \times \frac{1}{4}''$	0	10	8.08		27		
	2	Pin Pls.	$5'' \times \frac{3}{8}''$	0	11	6.38		11		
	2	" "	$6'' \times \frac{3}{8}''$	1	$1\frac{3}{4}$	7.65		18		
	4	Ls	$2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{4}''$	1	0	4.00		16		
	100	Lacing Bars	$1\frac{1}{2}'' \times \frac{1}{4}''$	1	0	1.28		128		
	264	Rivet Heads	$\frac{5}{8}''\phi$	per	100	9.95		26		
	84	" "	$\frac{3}{4}''\phi$	"	100	16.12		14		
							404	240	59.4	
				Total Weight of 4 Intermediate Posts = 644 × 2 =						1288
							Carried Forward			33560

1	2	3	4	5		6	7		8	9
Ref. No.	No. of Pieces	Shape	Section	Length		Wt. per Foot	Weight		Details Per Cent of Main Members	Total Weight
				Ft.	In.		Main Members	Details		
							Brought Forward			33568
5	4	Main Ties, U_1L_2, each thus:–								
	2	Eye Bars	$2\frac{1}{2}'' \times \frac{7}{8}''$	31	$3\frac{1}{16}$	7.44	465			
	4	Eyes	$2\frac{1}{2}'' \times \frac{7}{8}''$	1	$1\frac{1}{2}$	7.44		33		
							465	33	7.3	
				Total Weight of 4 Main Ties = 498 × 4 =						1992
	4	Main Ties, U_2L_3, each thus:–								
	2	Eye Bars	$2\frac{1}{2}'' \times \frac{3}{4}''$	31	$3\frac{1}{16}$	6.38	399			
	4	Eyes	$2\frac{1}{2}'' \times \frac{3}{4}''$	1	$1\frac{1}{2}$	6.38		29		
							399	29	7.3	
				Total Weight of 4 Main Ties = 428 × 4 =						1712
	4	Main Ties, U_3L_4, each thus:–								
	2	Eye Bars	$2'' \times \frac{1}{2}''$	31	$3\frac{1}{16}$	3.40	213			
	4	Eyes	$2'' \times \frac{1}{2}''$	1	$0\frac{1}{2}$	3.40		14		
							213	14	7.0	
				Total Weight of 4 Main Ties = 227 × 4 =						908
6	4	Hip Verticals, U_1L_1, each thus:–								
	2	[s	6" @ 8#	6	$5\frac{1}{2}$	8.00	104			
	2	Bars	$\frac{7}{8}''$ □	19	$7\frac{3}{4}$	2.60	102			
	2	Loops	$\frac{7}{8}''$ □	1	$6\frac{3}{8}$	2.60		8		
	2	"	$\frac{7}{8}''$ □	1	$8\frac{1}{4}$	2.60		9		
	4	Pin Pls.	$6'' \times \frac{1}{4}''$	0	$8\frac{1}{2}$	5.10		14		
	6	Batten Pls.	$5'' \times \frac{1}{4}''$	1	$1\frac{1}{4}$	4.25		28		
	4	Ls	$2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{4}''$	1	0	4.10		16		
	48	Rivet Heads	$\frac{5}{8}''$ φ	per	100	9.95		5		
	84	" "	$\frac{3}{4}''$ φ	"	100	16.12		14		
							206	94	45.6	
				Total Weight of 4 Hip Verticals = 300 × 4 =						1200
7	4	Counter Ties, U_3L_2, each thus:–								
	1	Rod	$\frac{3}{4}''$ φ	31	$3\frac{1}{16}$	1.50	47			
	1	Loop	$\frac{3}{4}''$ φ	1	6	1.50		2		
	1	"	$\frac{3}{4}''$ φ	1	$7\frac{3}{4}$	1.50		2		
	1	Turnbuckle			each	3.50		4		
	2	Upsets	1" φ	0	4	1.50		1		
							47	9	19.2	
				Total Weight of 4 Counter Ties = 56 × 4 =						224
	4	Counter Ties, U_4L_3, each thus:–								
	2	Rods	1" □	31	$3\frac{1}{16}$	3.40	213			
	2	Loops	1" □	1	$6\frac{7}{8}$	3.40		10		
	2	"	1" □	1	$8\frac{3}{4}$	3.40		11		
	2	Turnbuckles			each	7.00		14		
	2	Upsets	$1\frac{1}{2}''$ φ	0	4	3.40		2		
							213	37	17.5	
				Total Weight of 4 Counter Ties = 250 × 4 =						1000
							Carried Forward			40604

1	2	3	4	5		6	7		8	9
Ref. No.	No. of Pieces	Shape	Section	Length		Wt. per Foot	Weight		Details Per Cent of Main Members	Total Weight
				Ft.	In.		Main Members	Details		
							Brought Forward			40604
8	2	Floor beams, L_1, each thus:–								
	1	I	15″ @ 50#	16	5	50.00	820			
	4	Pls.	12″ × $\frac{1}{4}$″	0	$11\frac{3}{4}$	10.20		40		
	4	Std. Pls.	6″ × $\frac{7}{16}$″	1	6	8.93		54		
	64	Rivet Heads	$\frac{3}{4}$″ φ	per	100	16.12		10		
							820	104	12.3	
				Total Weight of 2 Floorbeams = 924 × 2 =						1848
	5	Floorbeams, $2L_2$, $2L_3$, L_4, each thus:–								
	1	I	15″ @ 50#	16	5	50.00	820			
	4	Pls.	12″ × $\frac{1}{4}$″	0	$11\frac{3}{4}$	10.20		40		
	4	Std. Pls.	5″ × $\frac{3}{8}$″	1	3	6.38		32		
	64	Rivet Heads	$\frac{3}{4}$″ φ	per	100	16.12		10		
							820	82	0.7	
				Total Weight of 5 Floorbeams = 902 × 5 =						4510
9		Joists, thus:–								
	4	[s	7″ @ $9\frac{3}{4}$#	20	9	9.75	809			
	12	[s	7″ @ $9\frac{3}{4}$#	19	$11\frac{1}{2}$	9.75	2340			
	14	Is	7″ @ 15#	20	9	15.00	4358			
	42	Is	7″ @ 15#	19	$11\frac{1}{2}$	15.00	12600			
							20107		0.0	
						Total Weight of Joists =				20107
10		Hub Guard (Fence), thus:–								
	4	[s	5″ @ $6\frac{1}{2}$#	16	$6\frac{11}{16}$	6.50	435			
	4	[s	5″ @ $6\frac{1}{2}$#	17	$8\frac{3}{8}$	6.50	461			
	4	[s	5″ @ $6\frac{1}{2}$#	18	$11\frac{1}{16}$	6.50	494			
	36	[s	5″ @ $6\frac{1}{2}$#	19	$11\frac{3}{4}$	6.50	4680			
							6070		0.0	
						Total Weight of Fence =				6070
11	2	Wall Plates, thus:–								
	1	L	3″ × 3″ × $\frac{1}{2}$″	16	0	9.90	158			
	1	L	3″ × 3″ × $\frac{1}{2}$″	15	$8\frac{1}{2}$	9.90	155			
	2	Pls.	6″ × $\frac{1}{4}$″	0	9	5.10		8		
	20	Rivet Heads	$\frac{3}{4}$″ φ	per	100	16.12		3		
							313	11		
					Total Weight of 2 Wall Plates =					324
12	5	Top Lateral Struts, each thus:–								
	2	Ls	3″ × 2″ × $\frac{1}{4}$″	18	0	4.10	148			
	2	Ls	3″ × 2″ × $\frac{1}{4}$″	16	0	4.10	133			
	2	Pls.	10″ × $\frac{5}{16}$″	1	4	10.63		28		
	2	"	6″ × $\frac{5}{16}$″	1	$6\frac{1}{2}$	6.38		19		
	2	"	12″ × $\frac{5}{16}$″	0	8	12.75		17		
				Carried Forward			281	64		73463

1	2	3	4	5		6	7		8	9
Ref. No.	No. of Pieces	Shape	Section	Length		Wt. per Foot	Weight		Details Per Cent of Main Members	Total Weight
				Ft.	In.		Main Members	Details		
							Brought Forward.			73463
12	Continued:—									
				Brought Forward			281	64		
	22	Lacing Bars	$1\frac{1}{2}'' \times \frac{1}{4}''$	1	$2\frac{3}{4}$	1.28		35		
	96	Rivet Heads	$\frac{5}{8}''\phi$	per	100	9.95		10		
	90	" "	$\frac{3}{4}''\phi$	"	100	16.12		15		
							281	124	44.0	
				Total Weight of 5 Top Lateral Struts = 405 × 5 =						2025
12a	5	Sway Struts, each thus:—								
	2	Ls	$3\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{4}''$	16	$5\frac{1}{2}$	4.90	162			
	4	Ls	$3\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{5}{16}''$	1	$2\frac{1}{4}$	6.10		30		
	2	Pls.	$11'' \times \frac{3}{8}''\Delta$	1	$2\frac{1}{4}$	14.03		35		
	66	Rivet Heads	$\frac{3}{4}''\phi$	per	100	16.12		11		
							162	76	47.0	
				Total Weight of 5 Sway Struts = 238 × 5 =						1190
13		Top Lateral Rods, thus:—								
	4	Rods	$1\frac{1}{4}''\phi$	26	$4\frac{1}{2}$	4.17	440			
	8	Upsets	$1\frac{5}{8}''\phi$	0	$3\frac{7}{8}$	4.17		8		
	8	Nuts	$1\frac{5}{8}''$		each	3.00		24		
	4	Rods	$1\frac{1}{8}''\phi$	26	4	3.38	355			
	8	Upsets	$1\frac{1}{2}''\phi$	0	$3\frac{7}{8}$	3.38		7		
	8	Nuts	$1\frac{1}{2}''$		each	2.35		19		
	4	Rods	$1''\phi$	26	$2\frac{1}{2}$	2.67	270			
	8	Upsets	$1\frac{1}{4}''\phi$	0	$4\frac{3}{8}$	2.67		6		
	8	Nuts	$1\frac{1}{4}''$		each	1.34		11		
							1065	75	7.0	
			Total Weight of Top Lateral Rods =							1140
13a		Sway Rods, thus:—								
	10	Rods	$\frac{7}{8}''\phi$	16	0	2.04	326			
	20	Upsets	$1\frac{1}{4}''\phi$	0	5	2.04		17		
	20	Clevises			each	8.00		160		
	20	Pins	$1\frac{1}{2}''\phi$	0	$2\frac{1}{2}$	1.50		30		
							326	207	60.0	
				Total Weight of Sway Rods =						533
14		Bottom Lateral Rods, thus:—								
	4	Rods	$1\frac{3}{8}''\phi$	25	8	5.05	520			
	4	Loops	$1\frac{3}{8}''\phi$	↓	$3\frac{3}{8}$	5.05		25		
	4	Clevises			each	17.00		68		
	4	Upsets	$1\frac{3}{4}''\phi$	0	$3\frac{1}{2}$	5.05		5		
	4	Nuts	$1\frac{3}{4}''$		each	3.70		15		
	4	Rods	$1\frac{1}{4}''\phi$	25	9	4.17	430			
	8	Upsets	$1\frac{3}{4}''\phi$	0	4	4.17		11		
	8	Nuts	$1\frac{3}{4}''$		each	3.70		30		
				Carried Forward			950	154		78351

1	2									
Ref. No.	No. of Pieces	Shape	Section	Ft.	In.	per Foot	Main Members	Details	Percent of Main Members	Total Weight
							Brought Forward			78351
14	Continued:-									
				Brought Forward			950	154		
	4	Rods	$1\frac{1}{8}$"φ	25	9	3.38	540			
	8	Upsets	$1\frac{1}{2}$"φ	0	$3\frac{7}{8}$	3.38		7		
	8	Nuts	$1\frac{1}{2}$"		each	2.35		19		
	4	Rods	1"φ	25	8	2.67	275			
	8	Upsets	$1\frac{1}{4}$"φ	0	$4\frac{3}{8}$	2.67		8		
	8	Nuts	$1\frac{1}{4}$"		each	1.34		11		
	28	Washers			"	1.70		48		
							1573	247	16.0	
				Total Weight of Bottom Lateral Rods =						1820
15	2	Portals, each thus:—								
	1	L	$3\frac{1}{2}$	18	0	[illegible]	110			
	1	L	3	15	$11\frac{3}{4}$	6.10	97			
	2	L^s	$3\frac{1}{2}$	12	9	6.10	156			
	2	L^s	$3\frac{1}{2}$	11	3	6.10	137			
	4	L^s	3	5	4	6.10	130			
	2	L^s	$3\frac{1}{2}$	8	11	4.90	87			
	2	Pls.	15	1	$7\frac{1}{2}$	16.58		54		
	1	Pl.	11	1	$7\frac{5}{8}$	9.34		16		
	2	Pls.	10"×$\frac{1}{4}$"	1	$6\frac{1}{2}$	8.50		26		
	100	Rivet Heads	$\frac{3}{4}$"φ	per	100	16.12		16		
	[illegible]	Washers	2"φ		each	1.00		9		
							717	121	17.0	
				Total Weight of 2 Portals = 838×2=						1676
16		Chord Pins and Nuts; thus:—								
				1	0	27.13		109		
				0	$1\frac{1}{4}$	16.69		15		
					each	3.00		24		
				0	$8\frac{3}{4}$	27.13		81		
				0	$1\frac{1}{4}$	16.69		15		
					each	3.00		24		
				0	8	19.29		129		
				0	$1\frac{1}{4}$	10.68		22		
					each	2.50		50		
				1	$0\frac{3}{4}$	19.29		82		
				0	$1\frac{1}{4}$	10.68		9		
					each	2.50		20		
				1	$1\frac{1}{8}$	27.13		119		
				0	$1\frac{1}{4}$	16.69		14		
					each	3.00		24		
				1	$2\frac{3}{4}$	27.13		133		
				0	$1\frac{1}{4}$	16.69		14		
					each	3.00		24		
				Carried Forward				908		81847

1	2	3	4	5		6	7		8	9
Ref. No.	No. of Pieces	Shape	Section	Length		Wt. per Foot	Weight		Details Per Cent of Main Members	Total Weight
				Ft.	In.		Main Members	Details		
							Brought	Forward		81847
16	Continued:—									
				Brought Forward				908		
	2	Pins L_4	$3\frac{3}{16}''\phi$	1	4	27.13		72		
	4	Screws	$2\frac{1}{2}''\phi$	0	$1\frac{1}{4}$	16.69		7		
	4	Nuts	$2\frac{1}{2}''$		each	3.00		12		
	4	Pins M_1	$2\frac{11}{16}''\phi$	0	7	19.29		42		
	8	Screws	$2''\phi$	0	$1\frac{1}{4}$	10.68		9		
	8	Nuts	$2''$		each	2.50		20		
								1070		
				Total Weight of Chord Pins and Nuts						1070
17	2	Pedestals, Roller End, each thus:—								
	1	Base Pl.	$15''\times\frac{5}{8}''$	1	4	31.88		43		
	4	Bolster Pls.	$12''\times\frac{1}{2}''$	0	$10\frac{1}{4}$	20.40		34		
	1	Pin Pl.	$6''\times\frac{3}{8}''$	0	$9\frac{1}{2}$	7.65		6		
	2	Rods	$\frac{3}{4}''\phi$	1	$3\frac{3}{4}$	1.50		4		
	2	Bars	$1\frac{3}{4}''\times\frac{1}{2}''$	1	$3\frac{3}{16}$	2.98		8		
	1	Bed Pl.	$15''\times\frac{5}{8}''$	1	$10\frac{3}{4}$	31.98		61		
	4	Ls	$3''\times2\frac{1}{2}''\times\frac{1}{4}''$	1	3	4.50		23		
	1	Tap Bolt	$1\frac{5}{16}''\phi$	0	$2\frac{1}{2}$	4.60		1		
	2	Anchor Bolts	$1\frac{1}{8}''\phi$	1	3	3.38		9		
	180	Rivet Heads	$\frac{3}{4}''\phi$	per	100	16.12		13		
	6	Rollers	$2''\phi$	1	$3\frac{3}{4}$	10.68		83		
								285		
				Total Weight of 2 Pedestals = 285×2=						570
	2	Pedestals, Fixed End, each thus:—								
	1	Base Pl.	$15''\times\frac{5}{8}''$	1	8	31.88		53		
	4	Bolster Pls.	$12''\times\frac{1}{2}''$	0	$10\frac{1}{4}$	20.40		54		
	1	Pin Pl.	$6''\times\frac{3}{8}''$	0	$9\frac{1}{2}$	7.65		6		
	1	Tap Bolt	$1\frac{5}{16}''\phi$	0	$2\frac{1}{2}$	4.60		1		
	46	Rivet Heads	$\frac{3}{4}''\phi$	per	100	16.12		7		
								125		
				Total Weight of 2 Pedestals = 125×2=						250
18		Bolts for Lumber, thus:—				Weight				
	80	Bolts	$\frac{5}{8}''\phi$	1	2	1.46		117		
	144	"	$\frac{5}{8}''\phi$	0	6	0.78		81		
	80	Cast Washers	$\frac{3}{4}''\phi$	per	100	43.00		34		
	368	Cut "		"	100			28		
								260		
					Total Weight of Bolts =					260
19		Spikes, thus:—								
	2500	Spikes	40d	per	100	7.00		175		
					Total Weight of Spikes =					175
						Total Weight of Metal				84172

TABLE LXXIII.

SUMMARY OF WEIGHT OF METAL IN BRIDGE

Ref. No.	Member	Weights: Main Members	Weights: Details	Weights: Total	Details Per Cent of Main Members
1	End Posts	5072	1440	6512	28.5
2	Top Chords	9696	2616	12312	27.0
3	Lower Chords	6416	876	7292	13.6
4	Intermediate Posts	4744	2708	7452	57.5
5	Main Ties	4308	304	4612	7.0
6	Hip Verticals	824	376	1200	45.6
7	Counters	1040	184	1224	18.0
8	Floorbeams	5740	618	6358	16.8
12	Top Lateral Struts	1405	620	2025	44.0
12a	Sway Struts	810	380	1190	47.0
13	Top Lateral Rods	1065	75	1140	7.0
13a	Sway Rods	326	207	533	60.0
14	Bottom Lateral Rods	1573	247	1820	16.0
15	Portals	1434	242	1676	17.0
16	Chord Pins and Nuts		1070	1070	
17	Pedestals		820	820	
		44453	12783	57236	28.7
Total Weight of Metal in Bridge, exclusive of 9,10,11,18 & 19 = 57236 lbs.					
9	Joists	20107		20107	0.0
10	Fence	6070		6070	0.0
11	Wall Plates	313	11	324	1.2
18	Bolts for Lumber		260	260	
19	Spikes " "		175	175	
		26490	446	26936	1.7
Total Metal in Bridge		70943	13229	84172	18.6

TABLE LXXIV.

WEIGHT OF LUMBER

Ref. No.	Name of Piece	No. of Pieces	Cross Section	Length in Feet	Feet B.M.	Weight	
						Lbs. per Ft. B.M.	Total
1	Flooring, Oak	160	2½"×12"	16	6400	4.5	28800
2	Joists, (Steel)						
3	Nailing Pieces	30	4"×6"	16	960	4.5	4320
4	Hub Guards, (Steel).						
5	Felloe Guards	20	4"×6"	16	640	4.5	2880
6	Shims (Not used)						
7	Plank at End of Joist	2	2½"×12"	18	90	4.5	405
			Total Lumber		8090		36405

TABLE LXXV.

COMPARISON OF DETAILS OF THIS SPAN WITH TWO OTHER 160 FOOT HIGHWAY SPANS

Ref. No.	Parts of Bridge	This Bridge	160'×16' 9 Panel Inclined Chord	160'×14' 10 Panel Inclined Chord
1	Weight of Pins in Per Cent of Total Weight of Metal exclusive of Fence, Joists, etc.	1.87	2.69	1.93
2	Weight of Rivet Heads in Per Cent of Total Weight of Metal exclusive of Fence, Joists, etc.	3.45	4.30	3.36
3	Total Weight of Details in Per Cent of Main Members exclusive of Fence, Joists, etc.	28.7	33.1	25.7
4	Weight of Bottom Lateral System in Pounds per lineal foot of Span	11.4	9.0	9.3
5	Weight of Top Lateral System in Pounds per lineal foot of Span	41.0	28.5	37.0
6	Weight of Trusses in Pounds per lineal foot of Span exclusive of Fence and Pedestals	260.5	224.9	277.4
7	Weight of Lumber in Pounds per lineal foot of Span	227.0	208.4	216.1
8	Weight of Joists in Pounds per lineal foot of Span	125.7	124.9	110.3
9	Weight of Fence in Pounds per lineal foot of Span	38.0	23.6	25.5
10	Weight of Floorbeams in Pounds per lineal foot of Span	40.0	41.4	30.6
11	Weight of Wall Plates, Pedestals, Bolts, & Spikes in Pounds per lineal foot of Span	9.9	11.7	9.9
12	Total Weight of Bridge in Pounds per linea foot of Span c. to c. End Pins	753.6	672.4	716.1

ESTIMATE OF COST.

Cost of Steel.—The cost of the steel will be estimated at the mill at Pittsburg, Pa.

Bridge Exclusive of Joists and Fence.—The bridge is composed of beams, angles, bars, plates and pin rounds as given in the following table:

Shape.	Weight, Pounds.	Per Cent of Total Weight.	Cost per Pound, Cents.	Percentage of Pound Cost, Cents.
Beams and channels	20,942	36.6	1.70	0.623
Angles 3″ and under	3,843	6.7	1.60	0.102
Bars	23,044	40.2	1.40	0.563
Plates	8,171	14.3	1.65	0.238
Pin rounds	1,236	2.2	2.00	0.044
Total	57,236	100.0		1.570

Average cost of the steel at the mill	1.570	cts. per lb.
Waste in fabrication, 4 per cent	0.063	" "
Paint material	0.010	" "
Freight, Pittsburg to Chicago	0.165	" "
Average cost of the steel at the shop	1.808	" "

Joists and Fence.—The joists, fence and wall plates will take the rate of 1.70 cts. per lb. at the mill.

Average cost of the steel at the mill	1.700	cts. per lb.
Waste in fabrication, 2 per cent	0.034	" "
Paint material	0.010	" "
Freight, Pittsburg to Chicago	0.165	" "
Average cost of the steel at the shop	1.909	" "

Shop Cost of the Steel in the Bridge Exclusive of Fence, Joists, etc.—

Average cost of the steel at the shop	1.808	cts. per lb.
Shop cost, including drafting	0.800	" "
Total shop cost	2.608	" "
Freight, shop to Orrick, Mo	0.270	" "
Total cost of steel at Orrick, Mo	2.878	" "

Shop Cost of Joists, Fence, etc.—

Average cost of the steel at the shop	1.909	cts. per lb.
Shop cost, including drafting	0.250	" "
Total shop cost	2.159	" "
Freight, shop to Orrick, Mo	0.270	" "
Total cost of steel at Orrick, Mo	2.429	" "

Erection.—

Hauling 42 tons 4 miles @ 25 cts. per ton mile and 50 cts per ton for loading and unloading	$ 63.00
Falsework.—Thirty-two piles, 30 ft. long @ 12 cts. per foot	115.20
Driving 960 lineal feet piling @ 20 cts. per foot	192.00
Timber, 7,000 ft. B. M.—½ price—@ $10.00	70.00
Placing timber, 7,000 ft. B. M. @ $8.00	56.00
Labor, erecting, bolting and riveting, 40 days' labor @ $4.00	160.00
Transportation of men and tools from Kansas City	50.00
Labor, painting bridge, one coat, 6 days' labor @ $4.00	24.00
Labor, erecting floor lumber, 8,100 ft. B. M. @ $4.00	32.40
Total cost of erection	$ 762.60

Summary of Cost of Superstructure.—

Steel, 57,236 lbs. @ 2.878 cts. per lb	$1,647.25
Joists, Fence, etc., 26,491 lbs. @ 2.429 cts. per lb	643.46
Lumber, 8,100 ft. B. M. oak @ $30.00	243.00
Paint, 16 gallons @ $1.25	20.00
Bolts for the floor, 300 lbs. @ 3 cts	9.00
Spikes for the floor, 200 lbs. @ 3 cts	6.00
Cost of erection	762.60
Total cost of superstructure	$3,331.31
Contract price	3,900.00
Profit	$ 568.69

CHAPTER XXII.

The Calculation of the Efficiencies of the Members of a 160-ft. Span Steel Pin-connected Highway Bridge.

Introduction.—The calculation of the efficiencies of the members of this bridge are to be made under the "Specifications for Steel Highway Bridges," Class D_1, as given in Appendix I, using the alternate stresses in § 36a to § 42a. The shop plans of this bridge are given in Fig. 285.

CALCULATION OF STRESSES. *Dead and Live Load Stresses.*—The dead and live load stresses in the trusses were calculated by the method of sections and are given in Fig. 286.

The dead joint load per truss is $W = 7{,}536 \times 20/2 = 7{,}536$ lbs., taken as 7,500 lbs.

The live joint load per truss was $P = 1{,}440 \times 20/2 = 14{,}400$ lbs. All loads were assumed as carried by the loaded chords.

Wind Load Stresses.—The wind load on the upper chord was assumed as a dead load of 150 lbs. per lineal foot of truss, one-half or 75 lbs. per foot was assumed as carried by the sway bracing and the other half by the upper lateral system. This gave wind joint loads of 1,500 lbs. at each upper chord point except at U_1 and U_1', where the loads were 3,000 lbs. The chord stresses due to this loading are marked D. The stresses in the upper lateral system due to wind were calculated by the method of sections.

The loads transferred to the lower lateral system by the sway bracing produce bending in the vertical posts and add to the vertical loads. The vertical load is $1{,}500 \times 24/17 = 2{,}160$ lbs., which is applied at all the lower chord points except L_1 and L_1'. The stresses in the upper and lower chords of the truss were calculated by the method of sections and are marked D'.

The stresses in the sway bracing have been calculated on the assumption that the posts are fixed by the floorbeams and that the point of contra-flexure is half way between the lower chord pin and the sway strut.

The stresses in the portal were calculated on the assumption that

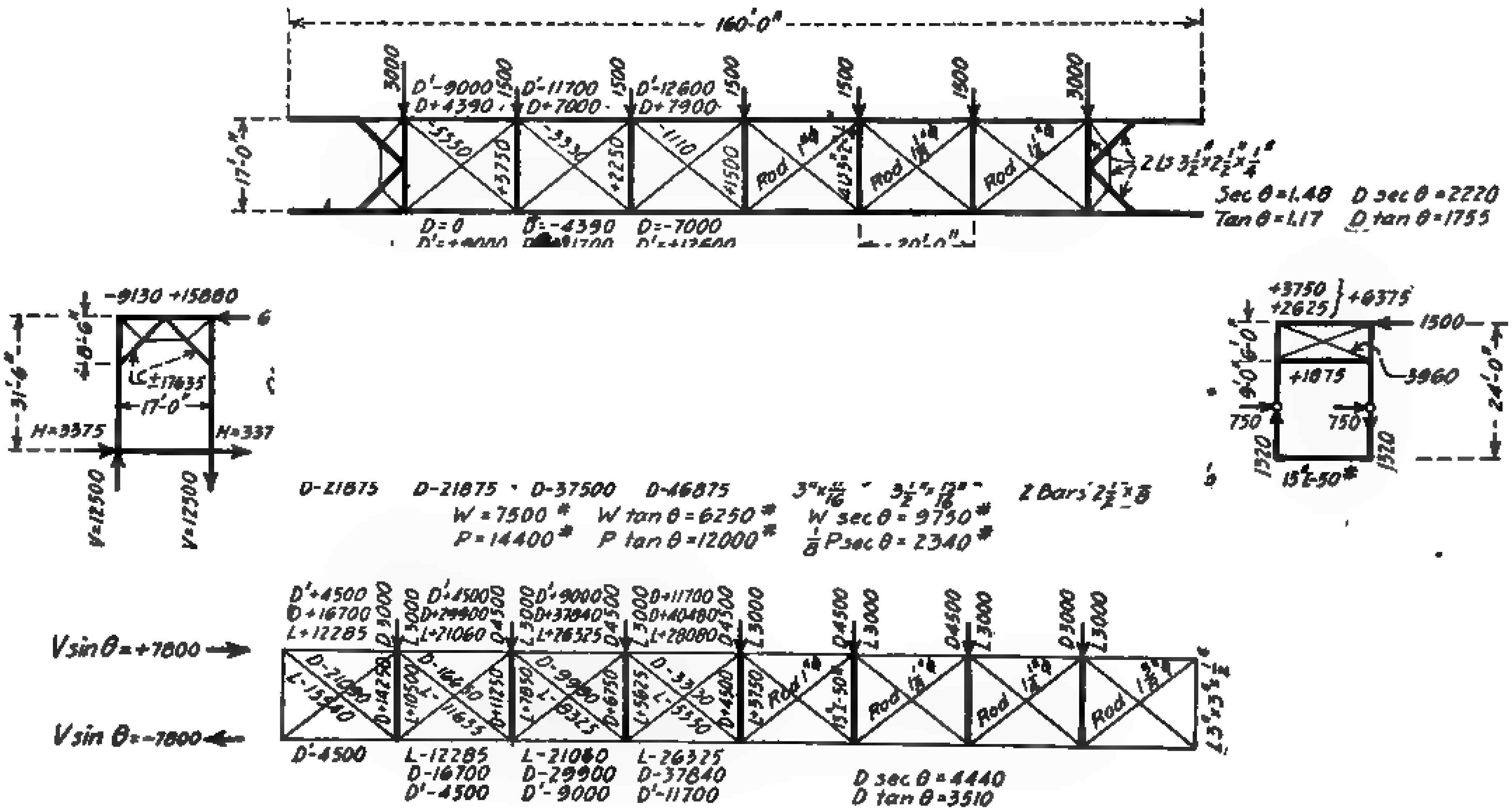


FIG. 286. STRESS DIAGRAM OF 160-FOOT SPAN HIGHWAY BRIDGE.

the end-posts were free to turn (pin-connected). The horizontal components of V and V' in the portal cause stresses which are the same in all panels of the lower chord, and are $+V\sin\theta$ on the windward side and $-V\sin\theta$ on the leeward side.

The wind loads on the lower lateral system were assumed to be a dead load of 150 lbs. per lineal foot and a live load of 150 lbs. per lineal foot. In addition to this the dead wind joint loads were increased by the 1,500 lbs. transferred by the sway bracing from the top lateral system, except at the points L_1 and L_1'. The dead and live load wind stresses are marked D and L, respectively.

INVESTIGATION OF EFFICIENCIES OF MEMBERS. VERTICAL POSTS. INTERMEDIATE POSTS U_2L_2.—Composed of 2 [s 7″ @ $9\frac{3}{4}$ lbs. × 24′ 0″ laced. From handbook, Cambria (p. 164), Carnegie (p. 101), the moment of inertia of one 7″ [@ $9\frac{3}{4}$ lbs. $= I_1 = 21.1''^4$; $r_1 = 2.72''$. Area of one 7″ [$= 2.85$ sq. in. Thickness of web$=0.21''$ (thickness should not be less than 0.25″).

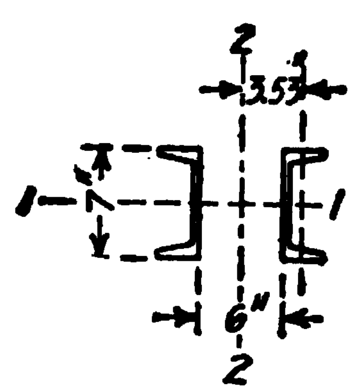


FIG. 287. INTERMEDIATE POST U_2L_2.

Moment of inertia I_2 may be calculated as follows:

$$I_2 = 2I' + 2A \times 3.53^2$$
$$= 1.96 + 2 \times 2.85 \times 3.53^2 = 72.98''^4$$
$$I_2 = A \cdot r_2^2;\ r_2 = 3.6''.$$

The largest ratio of l to r will occur around the axis 1–1, and $l/r = 288/2.72 = 106$.

The allowable live load unit stress (§ 37a), $P_L = 10{,}000 - 45 \cdot l/r = 5{,}230$ lbs.

The allowable dead load unit stress (§ 37a), $P_D = 20{,}000 - 90 \cdot l/r = 10{,}460$ lbs.

The maximum live load stress in the member $= +\ 27{,}000$ lbs.
The minimum live load stress in the member $= -\ 5{,}400$ lbs.
The dead load stress in the member $= +\ 11{,}250$ lbs.
There will be no reversal of stress in the member.

Area required to take the live load stress $= 27{,}000/5{,}230 = 5.16$ sq. in.
Area required to take the dead load stress $= 11{,}250/10{,}460 = 1.07$ sq. in.

Total required area for dead and live loads $= 6.23$ sq. in.
The actual area of the post is $2 \times 2.85 = 5.70$ sq. in.

(1) The efficiency of the post for dead and live load stresses $= 5.70/6.23 = 0.91$. The efficiency should be at least 1.00.

(2) The average allowable unit stress for dead and live loads is $38{,}250/6.23 = 6{,}140$ lbs.

The actual unit stress due to dead and live loads is $38{,}250/5.70 = 6{,}710$ lbs.

The efficiency of the post will also be the ratio of the allowable to the actual stress $= 6{,}140/6{,}710 = 0.91$. Either the first or second method may be used for calculating the efficiencies for direct stresses; for bending stresses the second method must be used.

Wind Load Stresses.—The direct wind load stress will be due to the half load of 1,500 lbs. transferred to the bottom chord by the sway bracing, the coefficient will be $2\frac{1}{2}$ and the stress $= 2\frac{1}{2} \times 2{,}160 = 5{,}400$ pounds.

Flexure in Post Due to Wind.—The maximum bending moment will come at the bottom of the sway strut, and from formula (63) will be

$$f_w = \frac{M_1 \cdot y_1}{I - \frac{P \cdot l^2}{24E}} = \frac{9 \times 12 \times 750 \times 5.09''}{72.98 - \frac{43{,}650 \times \overline{180}^2}{24 \times 28{,}000{,}000}} = 5{,}810 \text{ lbs.}$$

The extreme fiber stress will be $5{,}810 + 5{,}400/5.70 + (27{,}000 + 11{,}250)/5.70 = 13{,}470$ lbs.

When the wind is considered, the allowable unit stress for dead and live loads may be increased by 25 per cent if the wind load unit stress is more than 25 per cent of the unit stress due to dead and live loads (§ 46).

The allowable stress considering wind will be $6{,}140 \times 1.25 = 7{,}675$ lbs.
The efficiency considering wind will be $= 7{,}675/13{,}470 = 0.57$.
The efficiency should be at least 1.00.

INTERMEDIATE POST U_3L_3.—Composed of 2[s 7″ @ $9\frac{3}{4}$ lbs. laced $\times$ 24′ 0″.

This post has the same dimensions as post U_2L_2, and $I_1 = 21.1''^4$; $r_1 = 2.72''$; $A = 5.70$ sq. in.; $I_2 = 72.98''^4$; $r_2 = 3.6''$.

Also $l/r = 106$, $P_L = 5{,}230$ lbs., and $P_D = 10{,}460$ lbs.

The maximum live load stress in the member $= + 18{,}000$ lbs.
The minimum live load stress in the member $= 0$
The dead load stress in the member $= + 3{,}750$ lbs.
Area required to take the live load stress $= 18{,}000/5{,}230 = 3.44$ sq. in.
Area required to take the dead load stress $= 3{,}750/10{,}460 = 0.36$ sq. in.

Total required area to take dead and live loads $= 3.80$ sq. in.
The average allowable unit stress for dead and live loads is $21{,}750/3.80 = 5{,}700$ lbs.
The actual unit stress due to dead and live loads $= 21{,}750/5.7 = 3{,}810$ lbs.
The efficiency of the post for dead and live loads $= 5{,}700/3{,}810 = 1.50$.

Wind Load Stresses.—The direct wind load in the post will be $2{,}160 \times 1\frac{1}{2} = 3{,}240$ lbs. (The coefficient is $1\frac{1}{2}$.)

Flexure in Post Due to Wind.—The maximum bending moment will occur at the bottom of the sway strut, and will be

$$f_w = \frac{M_1 \cdot y_1}{I - \dfrac{P \cdot l^2}{24E}} = \frac{9 \times 12 \times 750 \times 5.09''}{72.98 - \dfrac{24{,}990 \times \overline{180}^2}{24 \times 28{,}000{,}000}} = 5{,}700 \text{ lbs.}$$

The extreme fiber stress will be

$$5{,}700 + 3{,}240/5.70 + (18{,}000 + 3{,}750)/5.70 = 10{,}090 \text{ lbs.}$$

When wind is considered the average allowable unit stress for dead and live loads may be increased 25 per cent (§ 46).
The allowable fiber stress considering wind will be $5{,}700 \times 1.25 = 7{,}125$ lbs.
The efficiency considering wind will be $7{,}125/10{,}090 = 0.71$.

INTERMEDIATE POST U_4L_4.—Composed of 2 [s 6″ @ 8 lbs., laced $\times$ 24′ 0″. From handbook the moment of inertia of one 6″ [$= I_1 = 13.0''^4$; $r_1 = 2.34''$. Area of one 6″ [$= 2.38$ sq. in. Thickness of web $= 0.20''$ (thickness should not be less than 0.25″).

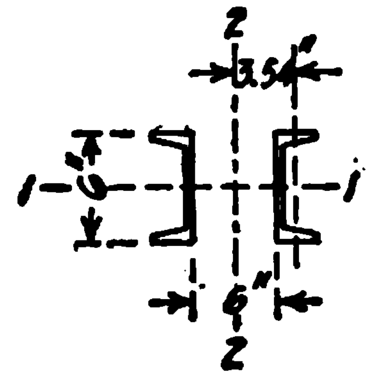


FIG. 288. INTERMEDIATE POST U_4L_4.

Moment of inertia, I_2, may be calculated as follows:

$$I_2 = 2I' + 2A \times \overline{3.54}^2 = 1.40 + 4.76 \times \overline{3.54}^2$$
$$= 61.04''^4$$
$$I_2 = Ar_2^2;\ r_2 = 3.58''$$

The largest ratio of l to r will occur around axis 1–1, and

$$l/r = 288/2.34 = 123.$$

The allowable live load unit stress (§ 37a), $P_L = 10{,}000 - 45 \cdot l/r = 4{,}465$ lbs.

The allowable dead load unit stress (§ 37a), $P_D = 20{,}000 - 90 \cdot l/r = 8{,}930$ lbs.

The maximum live load stress in the member $= +7{,}500$ lbs.

The dead load stress in the member $= 0$

Area required to take the live load stress $= 7{,}500/4{,}465 = 1.70$ sq. in.

Area required to take the dead load stress $= 0$.

Total area required for dead and live load stresses $= 1.70$ sq. in.

The average allowable unit stress for dead and live loads $= 7{,}500/1.70 = 4{,}465$ lbs.

The actual unit stress due to dead and live loads $= 7{,}500/1.70 = 1{,}570$ lbs.

The efficiency of the post for dead and live loads $= 4{,}465/1{,}570 = 2.80$.

Wind Load Stresses.—The direct wind load will be 2,160 lbs.

Flexure in Post Due to Wind.—The maximum bending moment will occur at the bottom of the sway strut and will be

$$f_w = \frac{M_1 \cdot y_1}{I - \frac{P \cdot l^2}{24E}} = \frac{9 \times 12 \times 750 \times 4.92''}{61.04 - \frac{9{,}660 \times 180^2}{24 \times 28{,}000{,}000}} = +6{,}740 \text{ lbs.}$$

The extreme fiber stress will be $6{,}740 + 2{,}160/4.76 + 7{,}500/4.76 = 8{,}770$ lbs.

When the wind is considered the average allowable unit stress for dead and live loads may be increased 25 per cent (§ 46).

The allowable fiber stress considering wind will be $4{,}465 \times 1.25 = 5{,}580$ lbs. The efficiency considering wind will be $= 5{,}581/8{,}770 = 0.64$.

END-POST L_0U_1.—The member is composed of 2 [s 10″ @ 15 lbs. and 1 pl. 12″ × ¼″, laced × 31′ $3\frac{1}{16}$″. Thickness of web $= 0.24''$ (should not be less than 0.25″).

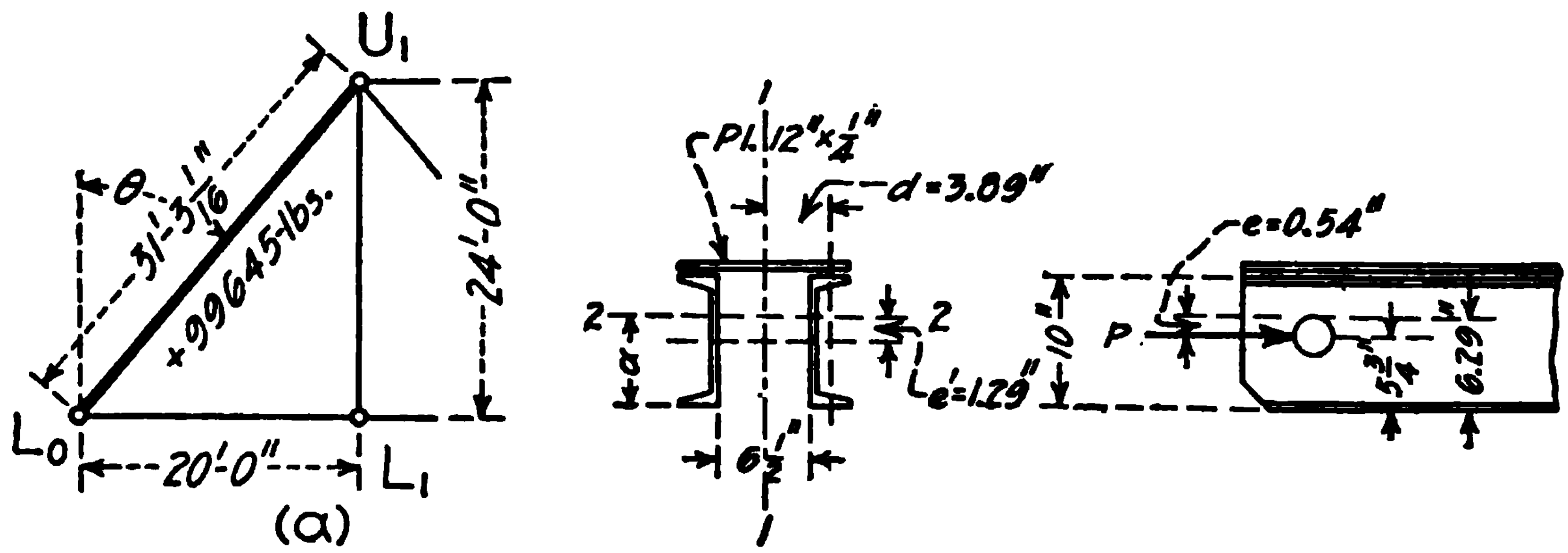


FIG. 289. END-POST L_0U_1.

Area 12" × ¼" Pl.	= 3.00 sq. in.	
Area 2 [s 10" @ 15 lbs.	= 8.92	"
Total area	= 11.92	"

To locate the neutral axis 2–2 take moments about the lower edge of the channels.

$$a = (8.92 \times 5 + 3 \times 10.125)/11.92 = 6.29''.$$

Eccentricity $e' = 6.29'' - 5.00'' = 1.29''$.

Now the pin is placed 5.75" from the lower edge of the channel and the eccentricity of the loading is $e = 6.29'' - 5.75'' = 0.54''$.

The moment of inertia, I_2, about axis 2–2 may be calculated as follows:

Let $I' = I$ of [s about the center of [s.
$I'' = I$ of plate about the center of the plate.
$A' =$ area of the [s.
$A'' =$ area of the plate.

$$\begin{aligned} \text{Then } I_2 &= I' + I'' + A' \times \overline{1.29}^2 + A'' \times \overline{3.835}^2 \\ &= 133.8 + 0.016 + 8.92 \times \overline{1.29}^2 + 3 \times \overline{3.835}^2 \\ &= 133.8 + 0.016 + 14.81 + 44.10 \\ &= 192.73''^4. \end{aligned}$$

$$I_2 = A \cdot r_2^2;\ r_2 = \sqrt{192.73/11.92} = 4.02''.$$

The moment of inertia, I_1, about axis 1–1 may be calculated as follows:

Let $I' = I$ of [s about the neutral axis parallel to the web.
$I'' = I$ of plate about axis 1–1.
$A' =$ area of [s.

Then $I_1 = I' + I'' + A' \times \overline{3.89}^2$
$= 4.60 + 36.0 + 8.92 \times \overline{3.89}^2$
$= 175.56''^4$.
$I_1 = A \cdot r_1^2;\ r_1 = \sqrt{175.56/11.92} = 3.84''$.

Direct Longitudinal Stress.—Length of member $= 31'\ 3\tfrac{1}{16}'' = 375''$.
Ratio of $l/r = 375/3.84 = 98$.

Maximum live load stress		$= + \ 65{,}520$ lbs.
Dead load stress		$= + \ 34{,}125$ lbs.
Direct wind stress	$= + 7{,}000 + 12{,}500$	$= + \ 19{,}500$ lbs.
Total direct stress		$= + 119{,}145$ lbs.

Allowable Stresses.—
For live loads (§ 37a), $P_L = 10{,}000 - 45 \cdot l/r = \ 5{,}590$ lbs.
For dead loads (§ 37a), $P_D = 20{,}000 - 90 \cdot l/r = 11{,}180$ lbs.
Area required to take the live load stress $= 65{,}520/5{,}590 \ = 11.72$ sq. in.
Area required to take the dead load stress $= 34{,}125/11{,}180 = \ 3.05$ sq. in.

Total required area for dead and live loads $= 14.77$ sq. in.
Average allowable unit stress for dead and live loads $= 99{,}645/14.77$ $= 6{,}760$ lbs.
Actual unit stress for dead and live loads $= 99{,}645/11.92 = 8{,}360$ lbs.
Efficiency for dead and live loads $= 6{,}760/8{,}360 = 0.81$.
The direct wind stress is less than 25 per cent of the allowable stresses for dead and live loads, and may be omitted in finding the efficiency for dead and live loads alone (§ 46).

Stress Due to Weight of Member.—The total weight of the member $= 1{,}628$ lbs. (See Chapter XXI.)
The bending moment due to the weight of the member, $M_1 = \tfrac{1}{8} W \cdot l \cdot \sin\theta$.
The stress due to weight is

$$f_w = \frac{M_1 \cdot y_1}{I - \dfrac{P \cdot l^2}{10E}}$$

The stress due to weight in the upper fiber is

$$f_w = \frac{\tfrac{1}{8} \times 1{,}628 \times 375 \times 0.645 \times 3.96''}{192.73 - \dfrac{99{,}645 \times 375^2}{10 \times 28{,}000{,}000}} = + \ 1{,}370 \text{ lbs.}$$

The stress due to weight in the lower fiber is

$$f_w' = -6.29/3.96 \times 1{,}370 = -2{,}180 \text{ lbs.}$$

Stress Due to Eccentric Loading.—The stress due to eccentric loading will be

$$f_e = \frac{M_1 \cdot y_1}{I - \frac{P \cdot l^2}{10E}} = \frac{P \cdot e \cdot y}{I - \frac{P \cdot l^2}{10E}}$$

The eccentric stress in the upper fiber will be

$$f_e = \frac{99{,}645 \times 0.54'' \times 3.96''}{192.73 - \frac{99{,}645 \times 375^2}{10 \times 28{,}000{,}000}} = -\ 1{,}490 \text{ lbs.}$$

Eccentric stress in lower fiber is

$$f_e' = +\ 6.29/3.96 \times 1{,}490 = +\ 2{,}370 \text{ lbs.}$$

The resultant stress due to the stress due to weight and the stress due to eccentric loading is $f_1 = f_w + f_e = +\ 1{,}370 - 1{,}490 = -\ 120$ lbs. in the upper fiber and $-\ 2{,}180 + 2{,}370 = +\ 190$ lbs. in the lower fiber.

The maximum stress in the member due to the direct loading, weight of member and eccentric loading occurs on the lower fiber, and is

$$f_2 + f_1 = 99{,}645/11.92 + 190 = 8{,}550 \text{ lbs.}$$

The resultant stress due to weight of the member and eccentric loading is less than 10 per cent, and may be neglected (§ 48).

Stress Due to Wind Moment.—The end-posts will both be fixed if the windward end-post is fixed. To fix the windward end-post the bending moment must not be greater than the resisting moment, which will be when

$$M_0 \leqq H \cdot y_0 \leqq (99{,}645 - V - D')a/2$$

If y_0 is taken equal to $\frac{1}{2}d = 11'\ 6'' = 138''$, and $a = 6.87''$, we will have

$$3{,}375 \times 138 \leqq (99{,}645 - 7{,}940 - 7{,}000)6.87/2$$

which makes $465{,}750 > 291{,}968$, and the windward post is not fixed.

The usual solution (Chapter VIII) would therefore assume that the end-posts are both pin-connected (free to turn at L_0), and the stress due to bending moment calculated on that assumption. In this case the maximum bending moment will occur at the foot of the portal knee brace, and will be $M_1 = 3{,}375 \times 276 = 931{,}500$ in.-lbs.

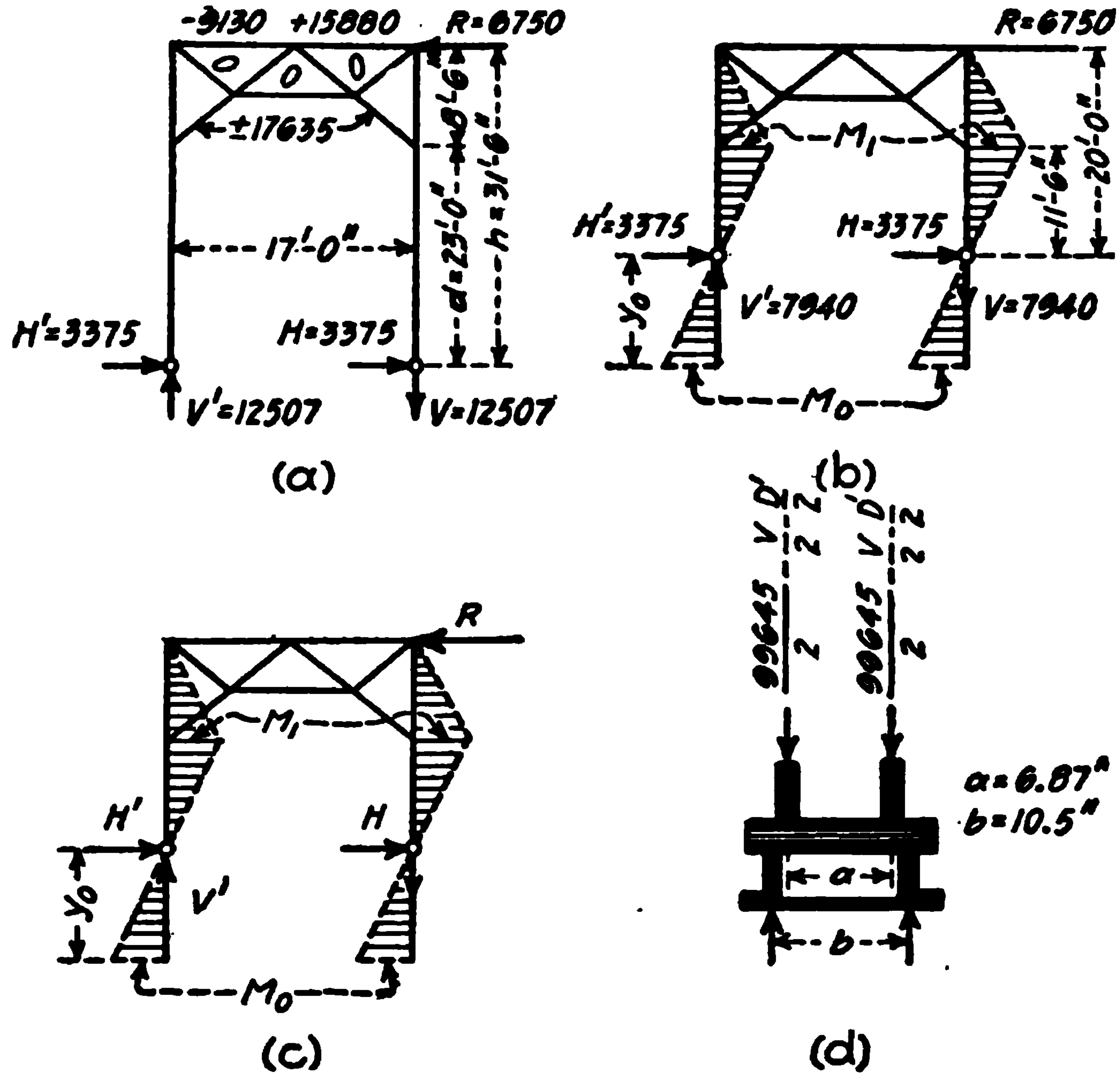


FIG. 290.

The following approximate solution appears to be more rational. The resisting moment at the foot of the windward end-post is

$$M_0 = (99{,}645 - 12{,}500 - 7{,}000)\tfrac{1}{2} \times 6.87''$$
$$= 275{,}280 \text{ in.-lbs.}$$

This will resist a bending moment of an equal amount. If y_0 is the distance from the foot of the post to the point of contra-flexure, we will have

$$H \cdot y_0 = 275{,}280 \text{ in.-lbs., and } y_0 = 275{,}280/3{,}375 = 81.5'' = 6'\ 9\tfrac{1}{2}''$$

The maximum bending moment will be M_1 and will occur at the foot of the portal knee brace, and will be

$$M_1 = 3{,}375(276 - 81.5) = 3{,}375 \times 194.5'' = 659{,}812 \text{ in.-lbs.}$$

which is only about two-thirds of the bending moment calculated above.

The stress due to wind moment in the windward post will be

$$f_w = \frac{M_1 \cdot y_1}{I - \frac{P \cdot l^2}{10E}} = \frac{659{,}812 \times 6''}{175.56 - \frac{(99{,}645 - 12{,}500 - 7{,}000)296.5^2}{10 \times 28{,}000{,}000}}$$

$= 26{,}210$ lbs. (Length $l = 378 - 81.5 = 296.5''$.)

The resisting moment at the foot of the leeward post will be

$$H \cdot y_0 = M_0 = (99{,}645 + 12{,}500 + 7{,}000)\tfrac{1}{2} \times 6.87'' = 409{,}260 \text{ in.-lbs.}$$

and $y_0 = 409{,}260/3{,}375 = 121'' = 10'\ 1''$

The bending moment at the foot of the knee brace on the leeward side will be $M_1 = 3{,}375\,(276 - 121)'' = 523{,}125$ in.-lbs.

The stress due to wind moment in the leeward end-post will be

$$f_w = \frac{M_1 \cdot y_1}{I - \frac{P \cdot l^2}{10E}} = \frac{523{,}125 \times 6''}{175.56 - \frac{(99{,}645 + 12{,}500 + 7{,}000)\,257^2}{10 \times 28{,}000{,}000}}$$

$= +\,21{,}200$ lbs. (Length $l = 378 - 121 = 257''$.)

Maximum Fiber Stresses.—The maximum fiber stresses due to wind and direct loading in the windward post will occur at the foot of the portal knee brace, and will be

$$f_1 = (65{,}520 + 34{,}125)/11.92 - 19{,}500/11.92 + 26{,}210$$
$$= +\,8{,}360 - 1{,}640 + 26{,}210 = +\,32{,}930 \text{ lbs.}$$

The maximum fiber stress due to wind and direct loading in the leeward post will occur at the foot of the portal knee brace, and will be

$$f_1 = (65{,}520 + 34{,}125)/11.92 + 19{,}500/11.92 + 21{,}200$$
$$= +\,8{,}360 + 1{,}640 + 21{,}200 = +\,31{,}200 \text{ lbs.}$$

The allowable stress when the wind is considered may be 25 per cent in excess of that allowed for dead and live loads $= 6{,}760 \times 1.25 = 8{,}450$ lbs. (§ 46).

The efficiency of the windward post $= 8{,}450/32{,}930 = 0.267$.

The efficiency of the leeward post $= 8{,}450/31{,}200 = 0.287$.

TOP CHORDS. TOP CHORD U_1U_2.—The member is composed of 2 [s 10″ @ 15 lbs. and 1 pl. 12″ × ¼″ × 20′ 0½″ long. The length is 240½″. Thickness of web $= 0.24''$ (thickness should not be less than $0.25''$). Properties of the sections are the same as for L_0U_1. Area $= 11.92$ sq. in.; $I_2 = 192.73''^4$; $r_2 = 4.02''$.
$I_1 = 175.56''^4$; $r_1 = 3.84''$; $e = 0.54''$.

Direct Longitudinal Stress.—Length of member $= 240\frac{1}{2}''$, and $l/r = 240.5/3.84 = 63$.

Maximum live load stress $= +72{,}000$ lbs.
Dead load stress $= +37{,}500$ lbs.
Direct wind load stress $= +\ 9{,}000$ lbs.

Allowable Stresses.—

For live loads (§37a), $P_L = 12{,}000 - 55 \cdot l/r = 8{,}535$ lbs.
For dead loads (§37a), $P_D = 24{,}000 - 110 \cdot l/r = 17{,}070$ lbs.
Area required to take live load stress $= 72{,}000/8{,}535 = 8.44$ sq. in.
Area required to take dead load stress $= 37{,}500/17{,}070 = 2.19$ sq. in.

Total area required for dead and live loads $= 10.63$ sq. in.
Average allowable unit stress for dead and live loads $= 109{,}500/10.63 = 10{,}300$ lbs.
Actual stress due to dead and live loads $= 109{,}500/11.92 = 9{,}200$ lbs.
Efficiency for dead and live loads $= 10{,}300/9{,}200 = 1.12$.
The direct wind load is less than 25 per cent of the dead and live stresses and may be neglected (§ 46).

Stress Due to Weight of Member.—The total weight of the member is 1,026 lbs (Chapter XXI). The bending moment is

$$M_1 = \tfrac{1}{8} W \cdot l = \tfrac{1}{8} \times 1{,}026 \times 240\tfrac{1}{2}''$$

$$f_w = \frac{M_1 \cdot y_1}{I - \dfrac{P \cdot l^2}{24E}} = \frac{\frac{1}{8} \times 1{,}026 \times 240.5'' \times 3.96''}{192.73 - \dfrac{109{,}500 \times 240.5^2}{24 \times 28{,}000{,}000}} = +\ 665 \text{ lbs.}$$

stress in the upper fiber.

The stress in the lower fiber is

$$f_w' = -\ 6.29/3.96 \times 665 = -\ 1{,}030 \text{ lbs.}$$

Stress Due to Eccentric Loading.—The eccentricity $= 0.54''$.

$$f_e = \frac{M_1 \cdot y_1}{I - \dfrac{P \cdot l^2}{24E}} = \frac{109{,}500 \times 0.54'' \times 3.96''}{192.73 - \dfrac{109{,}500 \times 240.5^2}{24 \times 28{,}000{,}000}} = -\ 1{,}220 \text{ lbs. in upper fiber.}$$

The stress in the lower fiber is

$$f_e = 6.29/3.96 \times 1{,}220 = +\ 1{,}940 \text{ lbs.}$$

The maximum compressive stress due to weight of member and eccentric loading is in the lower fiber, and is

$$f_1 = f_w + f_e = -1{,}030 + 1{,}940 = +910 \text{ lbs.}$$

This stress is less than 10 per cent of the allowable stress for dead and live loads and may be neglected (§ 48).

TOP CHORD U_2U_3.—The member is composed of 2 [s 10″ @ 15 lbs. and 1 pl. 12″ × ¼″ × 20′ 0½″ long. Length = 240½″. The thickness of the web = 0.24″ (thickness should not be less than 0.25″). The properties of the section are the same as for end-post L_0U_1.

Area = 11.92 sq. in.; $I_2 = 192.73''^4$; $r_2 = 4.02''$; $I_1 = 175.56''^4$; $r_1 = 3.84''$; $e = 0.54''$.

Direct Longitudinal Stress.—Length of member = 240½″, and $l/r = 240.5/3.84 = 63$.

Maximum live load stress = + 90,000 lbs.
Dead load stress = + 46,875 lbs.
Direct wind load stress = + 7,310 lbs.

Allowable Stresses.—

For live loads (§37a), $P_L = 12{,}000 - 55 \cdot l/r = 8{,}535$ lbs.
For dead loads (§ 37a), $P_D = 24{,}000 - 110 \cdot l/r = 17{,}070$ lbs.
Area required to take live load stress = 90,000/8,535 = 10.55 sq. in.
Area required to take dead load stress = 46,875/17,070 = 2.75 sq. in.

Total area required for dead and live loads = 13.30 sq. in.

The average allowable unit stress for dead and live loads = 136,875/13.30 = 10,300 lbs.

The actual unit stress due to dead and live loads = 136,875/11.92 = 11,400 lbs.

The efficiency of the member for dead and live loads = 10,300/11,400 = 0.90.

The direct wind load stress is less than 25 per cent of the allowable stresses for dead and live loads and may therefore be neglected (§ 46).

Stress Due to Weight of Member.—The total weight of the member = 1,026 lbs. (Chapter XXI). The bending moment, $M_1 = \frac{1}{8} W \cdot l = \frac{1}{8}\, 1026 \times 240.5''$,

$$f_w = \frac{M_1 \cdot y_1}{I - \frac{P \cdot l^2}{32E}} = \frac{\frac{1}{8} \times 1{,}026 \times 240.5 \times 3.96''}{192.73 - \frac{136{,}875 \times 240.5^2}{32 \times 28{,}000{,}000}} = +\,665 \text{ lbs.}$$

in the upper fiber.

Stress in the lower fiber is

$$f_w' = -\,6.29/3.96 \times 665 = -\,1{,}030 \text{ lbs.}$$

Stress Due to Eccentric Loading.—The eccentricity $= 0.54''$. The eccentric load $= 136{,}875 - 109{,}500 = 27{,}375$ lbs.

$$f_e = \frac{M_1 \cdot y_1}{I - \frac{P \cdot l^2}{32E}} = \frac{27{,}375 \times 0.54 \times 3.96''}{192.73 - \frac{136{,}875 \times 240.5^2}{32 \times 28{,}000{,}000}} = -\,320 \text{ lbs.}$$

in upper fiber.

The stress in the lower fiber is

$$f_e' = 6.29/3.96 \times 320 = +\,510 \text{ lbs.}$$

The maximum compressive stress due to weight of member and eccentric loading is in the upper fiber, and is

$$f_1 = f_w + f_e = +\,665 - 320 = +\,345 \text{ lbs.}$$

This stress is less than 10 per cent of the allowable stress for dead and live loads and may therefore be neglected (§ 48).

TOP CHORD U_3U_4.—The member is composed of 2 [s 10″ @ 15 lbs. and 1 pl. $12'' \times \frac{1}{4}'' \times 20'\ 0.\frac{1}{2}''$ long. Length of member $= 240.5''$. The thickness of the web $= 0.24''$ (thickness should not be less than $0.25''$). The properties of the section are the same as for the end-post L_0U_1.

Area $= 11.92$ sq. in.; $I_2 = 192.73''^4$; $r_2 = 4.02''$; $I_1 = 175.56''^4$; $r_1 = 3.84''$; $e = 0.54''$.

Direct Longitudinal Stress.—Length of member $= 240.5''$, and $l/r = 63$.

Maximum live load stress $= +\,96{,}000$ lbs.
Dead load stress $= +\,50{,}000$ lbs.
Direct wind load stress $= +\ \ 5{,}600$ lbs.

Allowable Stresses.—

For live loads (§37a), $P_L = 12{,}000 - \ 55 \cdot l/r = \ 8{,}535$ lbs.
For dead loads (§ 37a), $P_D = 24{,}000 - 110 \cdot l/r = 17{,}070$ lbs.

Area required to take live load stress $= 96{,}000/8{,}535 = 11.25$ sq. in.
Area required to take dead load stress $= 50{,}000/17{,}070 = 2.92$ sq. in.

Total required area for dead and live loads $= 14.17$ sq. in.

The average allowable unit stress for dead and live loads $= 146{,}000/14.17 = 10{,}300$ lbs.

The actual unit stress due to dead and live loads $= 146{,}000/11.92 = 12{,}200$ lbs.

The efficiency of the member for dead and live loads $= 10{,}300/12{,}200 = 0.84$.

The direct wind load stress is less than 25 per cent of the allowable stresses for dead and live loads, and therefore may be neglected (§ 46).

Stress Due to Weight of Member.—The total weight of the member $= 1{,}026$ lbs. (Chapter XXI).

$$f_w = \frac{M_1 \cdot y_1}{I - \dfrac{P \cdot l^2}{32E}} = \frac{\frac{1}{8} \times 1{,}026 \times 240.5'' \times 3.96''}{192.73 - \dfrac{146{,}000 \times 240.5^2}{32 \times 28{,}000{,}000}} = +665 \text{ lbs.}$$

in upper fiber.

The stress in the lower fiber due to weight of member is

$$f_w' = -6.29/3.96 \times 665 = -1{,}030 \text{ lbs.}$$

Stress Due to Eccentric Loading.—The eccentricity $= 0.54''$. The eccentric load $= 146{,}000 - 136{,}875 = 9{,}125$ lbs.

$$f_e = \frac{M_1 \cdot y_1}{I - \dfrac{P \cdot l^2}{32E}} = \frac{9{,}125 \times 0.54'' \times 3.96''}{192.73 - \dfrac{146{,}000 \times 240.5^2}{32 \times 28{,}000{,}000}} = -107 \text{ lbs.}$$

in upper fiber.

The stress in the lower fiber is

$$f_e' = 6.29/3.96 \times 107 = +170 \text{ lbs.}$$

The maximum fiber stress due to weight of the member and eccentric loading is in the upper fiber, and is

$$f_1 = f_w + f_e = +665 - 107 = +558 \text{ lbs.}$$

This stress is less than 10 per cent of the allowable stress for dead and live loads and may be neglected (§ 48).

TENSION MEMBERS. LOWER CHORD L_0L_1.—The member is composed of 2 eye-bars $2\frac{1}{2}'' \times \frac{7}{8}'' \times 20'\ 0''$. Total area $= 2'' \times 2.19'' = 4.38$ sq. in.

Longitudinal Stresses.—

Allowable live load unit stress (§ 36a)	$= 12,500$ lbs.
Allowable dead load unit stress (§ 36a)	$= 25,000$ lbs.
Maximum live load stress	$= -42,000$ lbs.
Dead load stress	$= -21,875$ lbs.
Maximum wind load compression	$= +41,285$ lbs.
Maximum wind load compression for dead loads	$= +29,000$ lbs.
Maximum wind load tension	$= -12,300$ lbs.

There is a reversal of stress for dead loads of $+29,000 - 21,875 = +7,125$ lbs. No provision is made for this reversal of stress, and the compression will have to be taken by the floor system.

Area required to take live load stress $= 42,000/12,500 = 3.36$ sq. in.
Area required to take dead load stress $= 21,875/25,000 = 0.88$ sq. in.

Total area required for dead and live loads 4.24 sq. in.

The average allowable unit stress for dead and live loads
$= 63,875/4.24 = 15,060$ lbs.

Actual unit stress due to dead and live loads $= 63,875/4.38 = 14,650$ lbs.
The efficiency for dead and live loads $= 15,060/14,650 = 1.03$.

Stress Due to Eccentricity.—None.

Stress Due to Weight.—From the diagram in Fig. 107 the stress due to weight is $f_w = 660$ lbs. per sq. in.

The stress due to weight of the member and eccentricity are less than 10 per cent of the allowable stress for dead and live loads, and may be neglected (§ 48).

Wind Load Stresses.—The maximum wind load tension is $-12,300$ lbs., which is less than 25 per cent of the allowable dead and live load stress and may be neglected (§ 46).

Lower chords L_0L_1 should be made stiff members.

LOWER CHORD L_1L_2.—The member is composed of 2 eye-bars $2\frac{1}{2} \times \frac{7}{8}'' \times 20'\ 0''$. This member has the same dimensions as L_0L_1, and the dead and live load stresses are the same. The wind load stresses, however, are different.

Longitudinal Stresses.—

Allowable live load unit stress (§ 36a)	$=12,500$ lbs.
Allowable dead load unit stress (§ 36a)	$=25,000$ lbs.
Maximum live load stress	$=-42,000$ lbs.
Dead load stress	$=-21,875$ lbs.
Maximum wind load compression	$=+63,260$ lbs.
Dead wind load compression	$=+42,200$ lbs.
Maximum wind load tension	$=-41,285$ lbs.

There is a reversal of stress for dead loads of $+42,200-21,875=+20,325$ lbs., while the maximum live and dead load stresses are nearly neutralized by the maximum wind load compression. No provision is made for reversal of stress, and the compression will either be taken by the floor system or will buckle the lower chord, producing arch action in the upper chord.

Area required to take live load stress $=42,000/12,500=3.36$ sq. in.
Area required to take dead load stress $=21,875/25,000=0.88$ sq. in.

Total area required for dead and live loads $=4.24$ sq. in.

The average allowable unit stress for dead and live loads
$=63,875/4.24=15,060$ lbs.

Actual unit stress for dead and live loads $=63,875/4.38=14,650$ lbs.
The efficiency for dead and live loads $=15,060/14,650=1.03$.

Stress Due to Weight.—From the diagram in Fig. 107 the stress due to weight is $f_w=660$ lbs. per sq. in.

Stress Due to Eccentric Loading.—None.

The stress due to weight and eccentric loading is less than 10 per cent of the allowable stress for dead and live loads, and may be neglected (§ 48).

Wind Load Stresses.—The maximum wind load tension is $-41,285$ lbs., and must be considered, since it is more than 25 per cent of the allowable stress for dead and live loads (§ 46).

The actual stress when wind is considered is

$$-42,000/4.38-21,875/4.38-41,285/4.38=-24,100 \text{ lbs.}$$

The allowable average unit stress for dead and live loads $=15,060$ lbs., to which may be added 25 per cent when wind loads are considered (§ 46) $=15,060\times 1.25=18,825$ lbs.

The efficiency of the member when wind is considered is

$$18,825/24,100=0.78$$

Lower chords L_1L_2 should be made stiff members.

LOWER CHORD L_2L_3.—The member is composed of 2 eye-bars $3\frac{1}{2}'' \times 1\frac{5}{8}'' \times 20'\ 0''$. The total area $= 2'' \times 3.28'' = 6.56$ sq. in.

Longitudinal Stresses.—

Allowable live load unit stress (§ 36a)	$= 12{,}500$ lbs.
Allowable dead load unit stress (§ 36a)	$= 25{,}000$ lbs.
Maximum live load stress	$= -72{,}000$ lbs.
Dead load stress	$= -37{,}500$ lbs.
Maximum wind load compression	$= +80{,}965$ lbs.
Maximum wind load compression for dead loads	$= +54{,}640$ lbs.
Maximum wind load tension	$= -67{,}760$ lbs.

There is a reversal of stress for dead load of $-37{,}500 + 54{,}640 = +17{,}140$ lbs. No provision is made for this reversal and the compression will be taken by the floor system, or the truss will act as an arch.

Area required to take live load stress $= 72{,}000/12{,}500 = 5.76$ sq. in.
Area required to take dead load stress $= 37{,}500/25{,}000 = 1.50$ sq. in.

Total area required for dead and live loads $= 7.26$ sq. in.
The average allowable unit stress for dead and live loads

$$= 109{,}500/7.26 = 15{,}060 \text{ lbs.}$$

The actual unit stress for dead and live loads $= 109{,}500/6.56 = 16{,}700$ lbs.
The efficiency for dead and live load stresses $= 15{,}060/16{,}700 = 0.90$.

Stress Due to Weight.—From the diagram in Fig. 107 the stress due to weight is $f_w = 800$ lbs. per sq. in.

Stress Due to Eccentric Loading.—None.

The stress due to the weight of the member and the eccentric loading of the member is less than 10 per cent of the allowable stress for dead and live loads, and may be neglected (§ 48).

Wind Load Stresses.—The maximum wind load tension $= -67{,}760$ lbs. and must be considered. The maximum unit stress due to wind and dead and live loads $= -67{,}760/6.56 - 37{,}500/6.56 - 72{,}000/6.50 = -27{,}000$ lbs. The allowable average stress for dead and live loads is 15,060 lbs., to which may be added 25 per cent when wind loads are considered $= 15{,}060 \times 1.25 = -18{,}825$ lbs. (§ 46).

The efficiency of the member when wind is considered is

$$= 18{,}825/27{,}000 = 0.70.$$

(It is not usual to make more than two panels of the lower chord stiff members. The compression due to dead load wind will doubtless be resisted by the floor system.)

LOWER CHORD L_3L_4.—The member is composed of 4 eye-bars $3'' \times 1\frac{1}{8}'' \times 20'\ 0''$. The total area $= 4'' \times 2.06'' = 8.24$ sq. in.

Longitudinal Stresses.—

Allowable live load unit stress (§ 36a)	$= 12{,}500$ lbs.
Allowable dead load unit stress (§ 36a)	$= 25{,}000$ lbs.
Maximum live load stress	$= -90{,}000$ lbs.
Dead load stress	$= -46{,}875$ lbs.
Maximum wind load compression	$= +88{,}060$ lbs.
Maximum wind load compression for dead loads	$= +59{,}980$ lbs.
Maximum wind load tension	$= -83{,}665$ lbs.

There is a reversal for dead loads of $-46{,}875 + 59{,}980 = +13{,}105$ lbs. No provision is made for this stress.

Area required to take live load stress $= 90{,}000/12{,}500 = 7.200$ sq. in.
Area required to take dead load stress $= 46{,}875/25{,}000 = 1.875$ sq. in.

Total area required for dead and live loads $= 9.075$ sq. in.

The average allowable unit stress for dead and live loads

$$= 136{,}875/9.075 = 15{,}060 \text{ lbs.}$$

The actual unit stress due to dead and live loads

$$= 136{,}875/8.24 = 16{,}600 \text{ lbs.}$$

The efficiency of the member for dead and live loads

$$= 15{,}060/16{,}600 = 0.90.$$

Stress Due to Weight.—From the diagram in Fig. 107 the stress due to weight is $f_w = 800$ lbs. per sq. in.

Stress Due to Eccentric Loading.—None.

The stress due to weight and eccentric loading is less than 10 per cent of the allowable stress for dead and live loads, and may be neglected (§ 48).

Wind Load Stress.—The maximum wind load tension $= -83{,}665$ lbs., which is more than 25 per cent of the allowable dead and live loads and must be considered (§ 46). The maximum unit stress due to wind, dead and live loads $= -83{,}365/8.24 - 46{,}875/8.24 - 90{,}000/8.24 = 26{,}750$ lbs. The average allowable unit stress for dead and live loads

is 15,060 lbs., to which 25 per cent may be added when wind loads are considered (§ 46) = 15,060 × 1.25 = − 18,825 lbs.

The efficiency of the member when wind is considered

$$= 18{,}825/26{,}750 = 0.70.$$

MAIN TIE U_1L_2.—The member is composed of 2 eye-bars $2\frac{1}{2}'' \times \frac{7}{8}'' \times 31'\ 3\frac{1}{16}''$. The total area $= 2'' \times 2.18'' = 4.36$ sq. in.

Longitudinal Stresses.—

Allowable live load unit stress (§ 36a)	= 12,500 lbs.
Allowable dead load unit stress (§ 36a)	= 25,000 lbs.
Maximum live load stress	= − 49,140 lbs.
Deal load stress	= − 24,375 lbs.
Minimum live load stress	= + 2,340 lbs.

The wind load stress is equal to 2,160/7,500 part of the dead load stress, which is less than 25 per cent of the dead and live load stress, and may be neglected (§ 46).

Area required to take live load stress = 49,140/12,500 = 3.93 sq. in.
Area required to take dead load stress = 24,375/25,000 = 0.98 sq. in.

Total area required for dead and live loads = 4.91 sq. in.

Average allowable unit stress for dead and live loads = 73,515/4.91 = 15,000 lbs.

Actual unit stress due to dead and live loads = 73,515/4.36 = 16,800 lbs.

The efficiency for dead and live loads = 15,000/16,800 = 0.90.

Stress Due to Weight.—From the diagram in Fig. 107 the stress due to weight is $f_w = 665 \times \sin\theta = 665 \times 0.645 = 429$ lbs.

Stress Due to Eccentric Loading.—None.

The stress due to weight is less than 10 per cent of the allowable stress for dead and live loads, and may be neglected (§ 48).

MAIN TIE U_2L_3.—The member is composed of 2 eye-bars $2\frac{1}{2}'' \times \frac{3}{4}'' \times 31'\ 3\frac{1}{16}''$. The area $= 2'' \times 1.88'' = 3.76$ sq. in.

Longitudinal Stresses.—

Allowable live load unit stress (§ 36a)	= 12,500 lbs.
Allowable dead load unit stress (§ 36a)	= 25,000 lbs.
Maximum live load stress	= − 35,100 lbs.
Dead load stress	= − 14,625 lbs.
Maximum live load compression	= + 7,020 lbs.

The wind load stress is nominal.

Area required to take live load stress $= 35,100/12,500 = 2.81$ sq. in.
Area required to take dead load stress $= 14,625/25,000 = 0.59$ sq. in.

Total area required for dead and live loads $= 3.40$ sq. in.

Average allowable unit stress for dead and live loads $= 49,725/3.40 = -14,600$ lbs.

Actual unit stress for dead and live loads $= 49,725/3.76 = -13,200$ lbs.

Efficiency for dead and live loads $= 14,600/13,200 = 1.06$.

Stress Due to Weight.—From the diagram in Fig. 107 the stress due to weight is $f_w = 850 \times \sin\theta = 850 \times 0.645 = 550$ lbs.

Stress Due to Eccentric Loading.—None.

The stress due to weight and eccentric loading is less than 10 per cent of the allowable stress for dead and live loads, and may be neglected (§ 48).

MAIN TIE U_8L_4.—The member is composed of 2 eye-bars $2'' \times \frac{1}{2}'' \times 31'\ 3\frac{1}{16}''$. The total area $= 2'' \times 1.00'' = 2.00$ sq. in.

Longitudinal Stresses.—

Allowable unit stress for live loads (§ 36a)	$= 12,500$ lbs.
Allowable unit stress for dead loads (§ 36a)	$= 25,000$ lbs.
Maximum live load stress	$= -23,400$ lbs.
Dead load stress	$= -4,875$ lbs.
Maximum live load compression	$= 0$

The wind load stress is nominal.

Area required to take live load stress $= 23,400/12,500 = 1.87$ sq. in.
Area required to take dead load stress $= 4,875/25,000 = 0.20$ sq. in.

Total area required for dead and live loads $= 2.07$ sq. in.

Average allowable stress for dead and live loads $= 28,275/2.07 = -13,700$ lbs.

Actual unit stress for dead and live loads $= 28,275/2.00 = 14,138$ lbs.

The efficiency for dead and live loads $= 13,700/14,138 = 0.92$.

Stress Due to Weight.—From the diagram in Fig. 107 the stress due to weight is $f_w = 650 \times \sin\theta = 650 \times 0.645 = 420$ lbs.

Stress Due to Eccentric Loading.—None.

The stress due to weight and eccentric loading is less than 10 per cent of the allowable stress for dead and live loads, and may be neglected (§ 48).

COUNTER TIE U_8L_2.—The member is composed of 1 rod $\frac{3}{4}''$ sq. $\times\ 31'\ 3\frac{1}{16}''$. This member carries no stress and need not be considered.

COUNTER TIE U_4L_8.—The member is composed of 2 bars 1" sq. × 31' $3\frac{1}{16}$". The total area = 2" × 1" = 2.00 sq. in.

Longitudinal Stresses.—

Allowable live load unit stress (§ 36a) = 12,500 lbs.
Allowable dead load unit stress (§ 36a) = 25,000 lbs.
Maximum live load stress = − 9,165 lbs.
Dead load stress = 0
Wind load stress = 0
Area required = 9,165/12,500 = 0.73 sq. in.
Allowable unit stress = 12,500 lbs.
Actual unit stress = 9,165/2.00 = 4,580 lbs.
Efficiency for dead and live loads = 12,500/4,580 = 2.74.

Stress Due to Weight.—From the diagram in Fig. 107 the stress due to weight is $f_w = 1{,}000 \times \sin\theta = 1{,}000 \times 0.645 = 645$ lbs.

Stress Due to Eccentric Loading.—None.

The stress due to weight and eccentric loading is less than 10 per cent of the allowable stress for dead and live loads, and may be neglected (§ 48).

HIP VERTICAL U_1L_1.—The member is made of two parts. The upper part is composed of 2 bars $\frac{7}{8}$" sq. × 19' $7\frac{3}{4}$" long; the lower part of 2[s 6" @ 8 lbs. × 6' $5\frac{1}{2}$" long.

A. UPPER PART.—The total area = 2" × 0.765" = 1.53 sq. in.

Longitudinal Stresses.—

Allowable unit stress for live loads (§36a) = 12,500 lbs.
Allowable unit stress for dead loads (§ 36a) = 25,000 lbs.
Maximum live load stress = − 14,400 lbs.
Dead load stress = − 7,500 lbs.
Wind load stress = 0
Area required to take live load stress = 14,400/12,500 = 1.15 sq. in.
Area required to take dead load stress = 7,500/25,000 = 0.30 sq. in.

Total area required for dead and live loads = 1.45 sq. in.
Average allowable unit stress for dead and live loads = 21,900/1.45 = 15,100 lbs.
Actual unit stress for dead and live loads = 21,900/1.55 = 14,300 lbs.
The efficiency for dead and live loads = 15,100/14,300 = 1.06.

Stress Due to Weight.—None.

Stress Due to Eccentric Loading.—None.

B. Lower Part.—The gross area of the section $=2'' \times 2.38'' = 4.76$ sq. in. The thickness of the web $= 0.20''$ (thickness should not be less than $0.25''$). The thickness of the flange at the center of the rivet hole $=$ (approximately) $(0.49'' + 0.20'')/2 = 0.35''$. Rivets are $\frac{3}{4}''$ diameter; holes $\frac{13}{16}''$ diameter; $\frac{7}{8}''$ diameter to be deducted in obtaining net area (Table XXXII).

The required net section for tension $= 1.45$ sq. in., the same as for the upper part. The net area through the pin hole must not be less than $1.45 \times 1.25 = 1.81$ sq. in. (Specifications, § 75).

Pin M_1.—Area through the pin $= (4.76 + 3.00) - (2\frac{11}{16} \times 2 \times 0.45) - (4 \times \frac{7}{8} \times 0.35) = 4.10$ sq. in., which is sufficient.

The required area back of the pin must not be less than 75 per cent of the required area through the pin (§75). The actual area back of the pin $= 2(3\frac{1}{4}'' - 1\frac{1}{32}'') \times 0.45'' = 2.00$ sq. in. Required area back of the pin $= 1.81'' \times 0.75'' = 1.38$ sq. in. The area back of the pin is sufficient.

Pin L_1.—The area through the pin hole $= (4.76 + 3.00) - 4(0.35 \times \frac{1}{2}) - (2\frac{11}{16} \times 2 \times 0.45) = 5.21$ sq. in., which is sufficient. The area back of the pin is more than sufficient.

CHORD PINS. PIN L_0.—The pin is $3\frac{3}{16}''$ diameter; grip $12''$; length $14\frac{1}{2}''$.

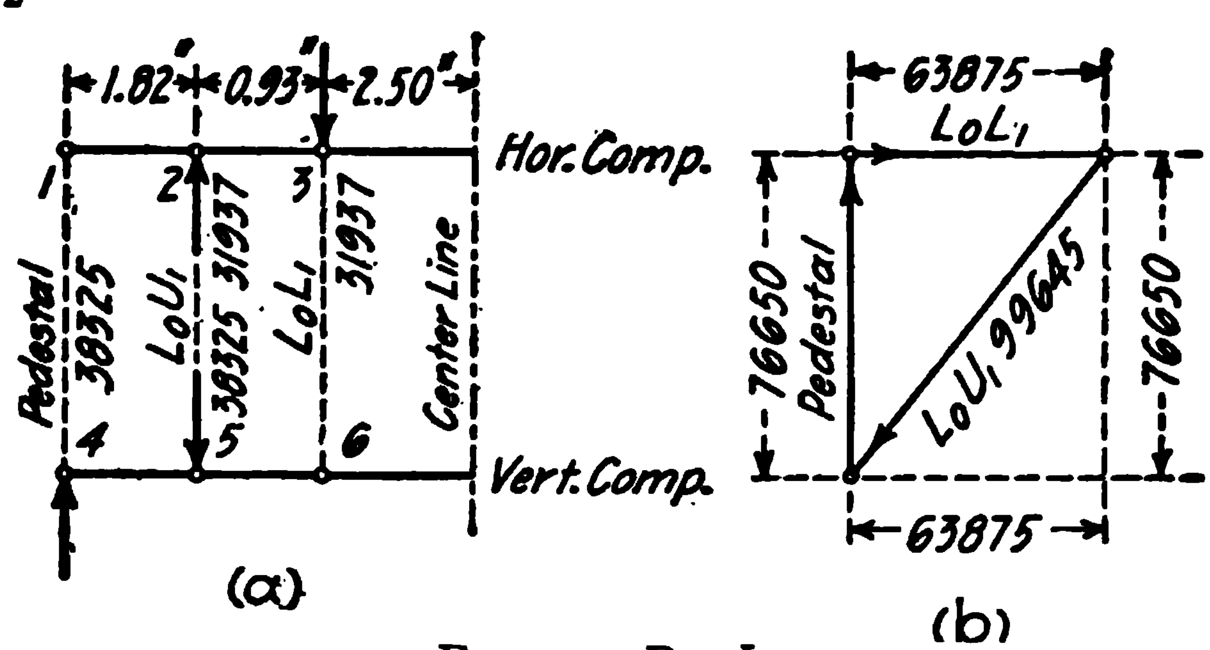


FIG. 291. PIN L_0.

Bending Moment. (a) *Not Considering Wind Stress.*—In Fig. 291 the stresses in (b) are the total dead and live load stresses, while the stresses in (a) are the stresses taken by the metal on one side of the member only. For an explanation of the method of calculation of the stresses in pins, see Chapter VIII.

Horizontal Components.—
Bending moment at $2 = 0$.

Bending moment at $3 = 31{,}937 \times 0.93'' = 29{,}700$ in.-lbs.

Vertical Components.—

Bending moment at $5 = 38{,}325 \times 1.82'' = 69{,}950$ in.-lbs.

Bending moment at $6 = 38{,}325 \times 2.75'' - 38{,}325 \times 0.93'' = 69{,}950$ in.-lbs.

The total moment at 2 and $5 = 69{,}950$ in.-lbs.

The total moment at 3 and $6 = \sqrt{\overline{29{,}700}^2 + \overline{69{,}950}^2} = 76{,}000$ in.-lbs.

The allowable bending moment for a fiber stress of 20,000 lbs. per sq. in. $= M = S \cdot I/c = (20{,}000 \times {}^1\!/\!_{64} d^4)/(d/2) = 63{,}650$ in.-lbs. (Table XXXIV).

The efficiency for dead and live loads $= 63{,}650/76{,}000 = 0.837$.

(b) *Considering Wind Stresses.*—Pin L_0 in the leeward end-post will have maximum stresses, and will be investigated.

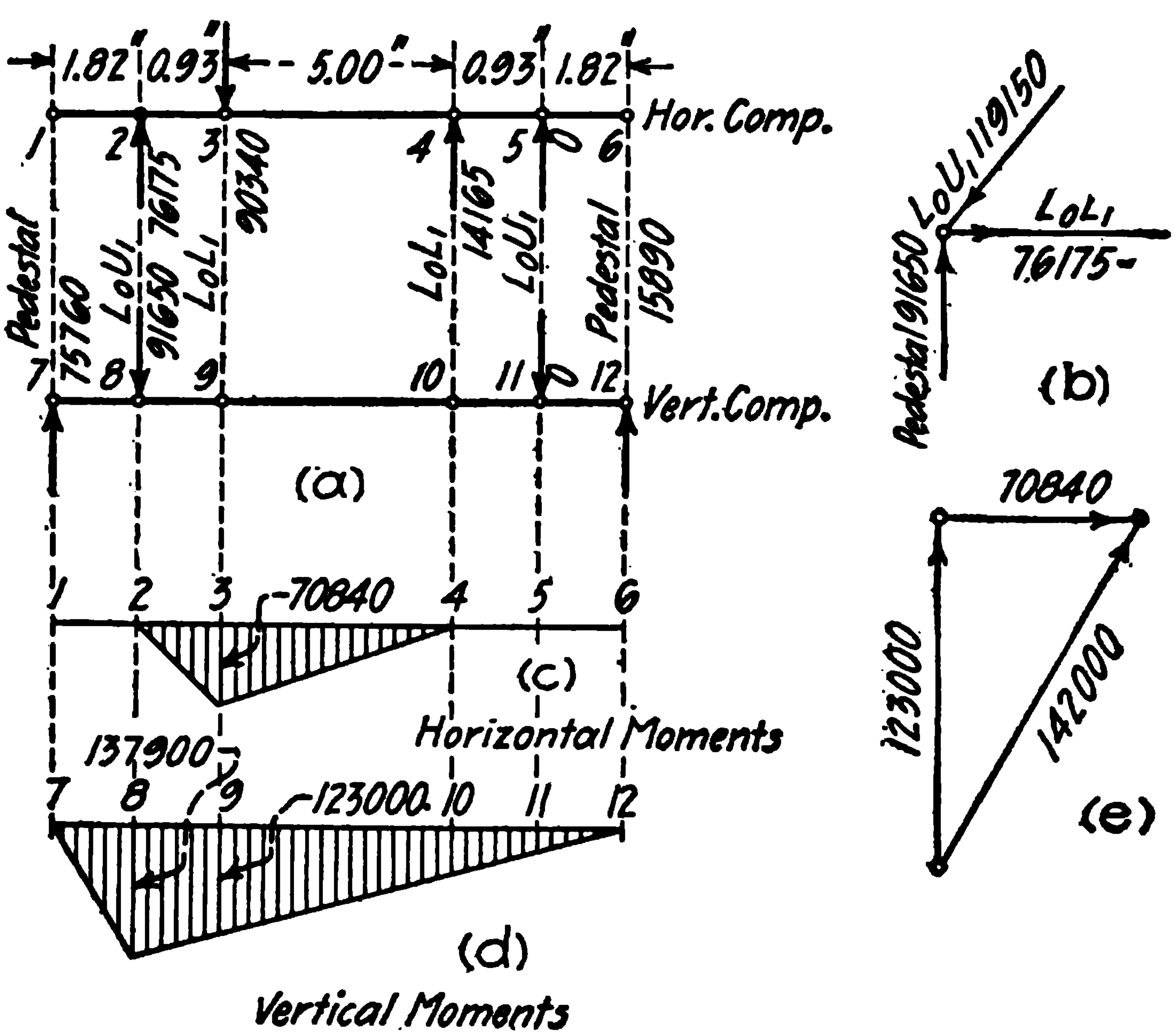


FIG. 292. PIN L_0. WIND LOAD STRESSES.

In Fig. 292 the stress in $L_0U_1 = +65{,}520 + 34{,}135 + 12{,}500 + 7{,}000 = 119{,}150$ lbs. The stress in $L_0L_1 = -42{,}000 - 21{,}875 - 7{,}800 - 4{,}500 = -76{,}175$ lbs. The pedestal reaction $= 91{,}650$ lbs. It is necessary to consider the entire length of the pin as shown in Fig. 292.

From Fig. 290 in the discussion of the end-post it will be seen that when $y_0 = 6' \, 9\frac{1}{2}''$, the resisting moment $M_0 =$ the bending moment M_1, which requires that the end-post L_0U_1 have a zero bearing on the windward side of the member, points 5 and 11. The total stress in L_0U_1 will then be carried at points 2 and 8, and will be as shown in Fig. 292.

Taking moments about 12, Fig. 292, we get the vertical pedestal reaction at $7 = 75{,}760$ lbs., and taking moments about 7 the pedestal reaction at $12 = 15{,}890$ lbs. Taking moments about 4 the stress in L_0L_1 at $3 = 90{,}340$ lbs., and taking moments about 3 the stress in L_0L_1 at $4 = 14{,}165$ lbs., the stresses acting in opposite directions, as shown.

Horizontal Components.—

Bending moment at $2 = 0$.

Bending moment at $3 = 76{,}175 \times 0.93'' = 70{,}840$ in.-lbs.

Bending moment at $4 = 76{,}175 \times 5.93'' - 90{,}340 \times 5.00'' = 0$ in.-lbs.

Vertical Components.—

Bending moment at $8 = 75{,}760 \times 1.82'' = 137{,}900$ in.-lbs.

Bending moment at $9 = 75{,}760 \times 2.75'' - 91{,}650 \times 0.93'' = 123{,}000$ in.-lbs.

Bending moment at $12 = 0$.

The maximum bending moment is at point 3 and 9

$$= \sqrt{70{,}840^2 + 123{,}000^2} = 142{,}000 \text{ in.-lbs.}$$

When the wind stress is considered the allowable bending moment for dead and live loads may be increased 25 per cent (§ 46) $= 63{,}650 \times 1.25 = 79{,}560$ in.-lbs.

The efficiency considering wind $= 79{,}560/142{,}000 = 0.56$.

Note.—The pin is too small for dead and live loads. It should have been at least $3\frac{5}{8}''$ diameter and the channels should have been spaced farther apart, *i. e.,* the top cover plate is too narrow.

The windward eye-bar in L_0L_1 will carry a dead, live and wind load $= 90{,}340$ lbs., which will give a fiber stress of 41,200 lbs. per sq. in., which is unsafe. This solution does not consider the rigidity of the joint, which is certainly an element of strength.

Bearing on Pin L_0.—(1) *Bearing of the End-post L_0U_1 on the pin.* (a) *Not considering wind.*

Bearing area for 2 $\frac{3}{8}''$ pl. $= 2 \times \frac{3}{8}'' \times 3\frac{3}{16}''$	$= 2.40$ sq. in.
Bearing area for 2 $\frac{1}{4}''$ pl. $= 2 \times \frac{1}{4}'' \times 3\frac{3}{16}''$	$= 1.60$ sq. in.
Bearing area for 2 10″ [s @ 15 lbs. $= 2 \times 0.24'' \times 3\frac{3}{16}''$	$= 1.53$ sq. in.
Total bearing area	$= 5.53$ sq. in.

The dead and live load stress in $L_0U_1 = 99{,}645$ lbs.

The unit bearing stress $= 99{,}645/5.53 = 18{,}050$ lbs. The allowable unit bearing stress $= 18{,}000$ lbs. (§ 40a).

Efficiency for dead and live loads $= 18{,}000/18{,}050 = 0.998$.

(b) *Considering Wind.*—The total stress comes on one-half of the bearing area and unit stress $= 119{,}500/2{,}765 = 43{,}200$ lbs. The allowable unit bearing stress for wind stress and dead and live load stresses $= 18{,}000 \times 1.25 = 22{,}250$ lbs. (§ 46).

Efficiency when wind loads are considered $= 22{,}250/43{,}200 = 0.50$.

(1) *Bearing of the Pedestal on Pin L_0.* (a) *Not Considering Wind.*—Bearing area $= 4 \times \frac{1}{2}'' \times 3\frac{3}{16}'' = 6.37$ sq. in. The stress in the pedestal $= 76{,}650$ lbs.

Actual bearing stress $= 76{,}650/6.37 = 12{,}040$ lbs.

Allowable bearing unit stress $= 18{,}000$ lbs.

Efficiency for dead and live loads $= 18{,}000/12{,}040 = 1.50$.

(b) *Considering Wind.*—The maximum stress is on the leeward side $= 75{,}650$ lbs.

Actual unit bearing stress $= 75{,}650/3.18 = 24{,}000$ lbs.

Allowable bearing stress considering wind $= 18{,}000 \times 1.25 = 22{,}500$ lbs. (§ 46).

Efficiency considering wind $= 22{,}500/24{,}000 = 0.936$.

(3) *Bearing of Lower Chord L_0L_1.* (a) *Not Considering Wind.*—The member is composed of 2 eye-bars $2\frac{1}{2}'' \times \frac{7}{8}'' \times 20'\ 0''$.

The bearing area $= 2 \times \frac{7}{8}'' \times 3\frac{3}{16}'' = 5.58$ sq. in.

The maximum stress $= 63{,}875$ lbs. The actual unit bearing stress $= 63{,}875/5.58 = 10{,}900$ lbs.

Allowable unit bearing stress $= (14{,}650 \times 4/3) = 19{,}500$ lbs. (§ 76).

The efficiency $= 19{,}500/10{,}900 = 1.80$.

The specifications, § 76, specify that pins must have a diameter at least equal to $\frac{3}{4}$ of the depth of the largest bar. This member satisfies this specification.

(b) *Considering Wind.*—The maximum bearing when wind is considered $= 90{,}340$ lbs. Actual unit bearing stress $= 90{,}340 \div 2.79 = 32{,}400$ lbs. The allowable bearing stress when wind is considered $= 19{,}500 \times 1.25 = 24{,}375$ lbs. The bearing is not sufficient when wind is considered.

Shear in Pin L_0. (a) *Not Considering Wind* (Fig. 291).

Horizontal Components.—

Shear between 1 & 2 = 0.

Shear between 2 & 3 = 31,937 lbs.
Shear to the right of 3 = 0.

Vertical Components.—
Shear between 4 & 5 = 38,735 lbs.
Shear between 5 & 6 = 0.
The maximum shear is between 1 & 2 and 4 & 5, and = 38,375 lbs.
Allowable shear = area of $3\frac{3}{16}''$ pin × 10,000 = 7,979 × 10,000
= 79,790 lbs.
Efficiency for shear = 79,790/38,375 = 2.08.

(b) *Considering Wind* (Fig. 292).

Horizontal Components.—
Shear between 1 & 2 = 0 lbs.
Shear between 2 & 3 = 76,175 lbs.
Shear between 3 & 4 = 14,165 lbs.

Vertical Components.—
Shear between 7 & 8 = 75,760 lbs.
Shear between 8 & 12 = 15,890 lbs.
The maximum shear is between 2 & 3 and 8 & 12 and

$$= \sqrt{\overline{76,175}^2 + \overline{15,890}^2} = 77,000 \text{ lbs.}$$

The allowable shear for wind = 79,790 × 1.25 = 99,740 lbs.
Efficiency = 99,740/77,000 = 1.29.

PIN U_1.—The pin is $3\frac{3}{16}''$ diameter; grip $8\frac{3}{4}''$; length $11\frac{1}{4}''$.

Bending Moment. (a) *Maximum Stress in End-post L_0U_1 and Top Chord U_1U_2.*—The stresses in the members are given in (b) Fig.

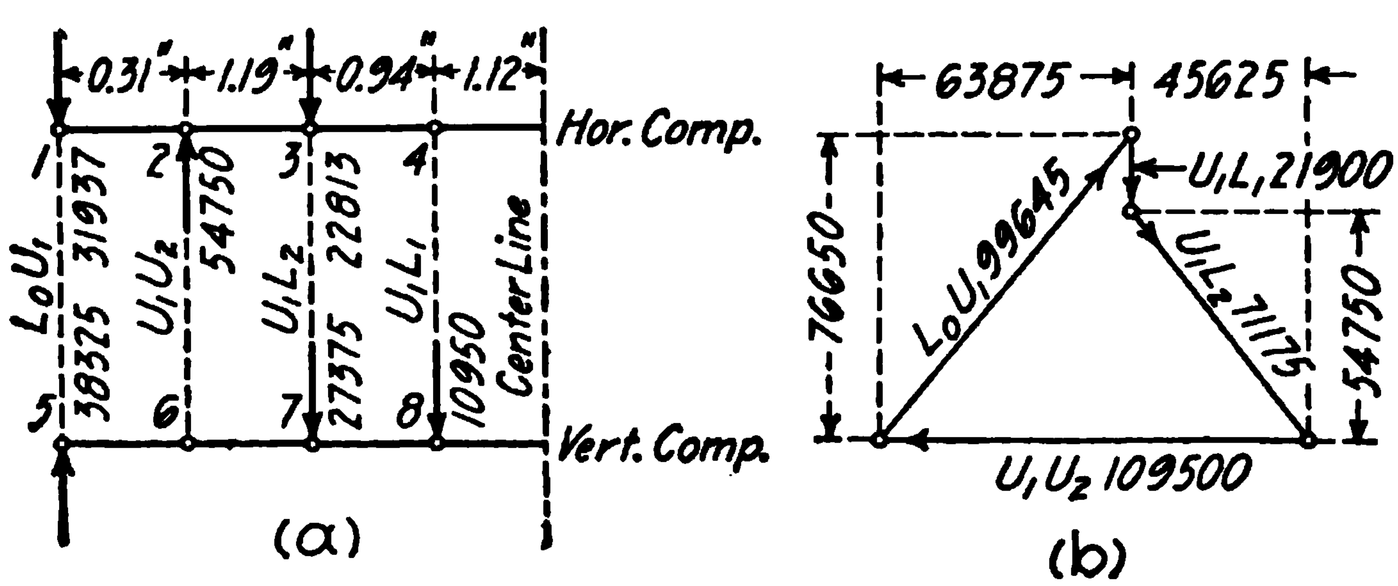


FIG. 293. PIN U_1. MAXIMUM STRESSES IN L_0U_1 AND U_1U_2.

293, while the horizontal and vertical components and the distances between centers of the metal composing the members are given in (a) Fig. 293 for the left half of the pin.

Horizontal Components.—
Bending moment at 1 = 0.
Bending moment at 2 = 31,937 × 0.31″ = 9,900 in.-lbs.
Bending moment at 3 & 4 = 31,937 × 1.50″ − 54,750 × 1.19″
= 33,960 in.-lbs.

Vertical Components.—
Bending moment at 5 = 0.
Bending moment at 7 = 38,325 × 1.50″ = 57,790 in.-lbs.
Bending moment at 8 = 38,325 × 2.44″ − 27,375 × 0.94″ = 67,770 in.-lbs.

Maximum bending moment is at 4 and 8 and $= \sqrt{33,960^2 + 67,770^2}$
= 75,800 in.-lbs.
The allowable bending moment for a $3\frac{15}{16}$″ pin (Table XXXIV)
= 63,650 in.-lbs.
Efficiency = 63,650/75,800 = 0.84.

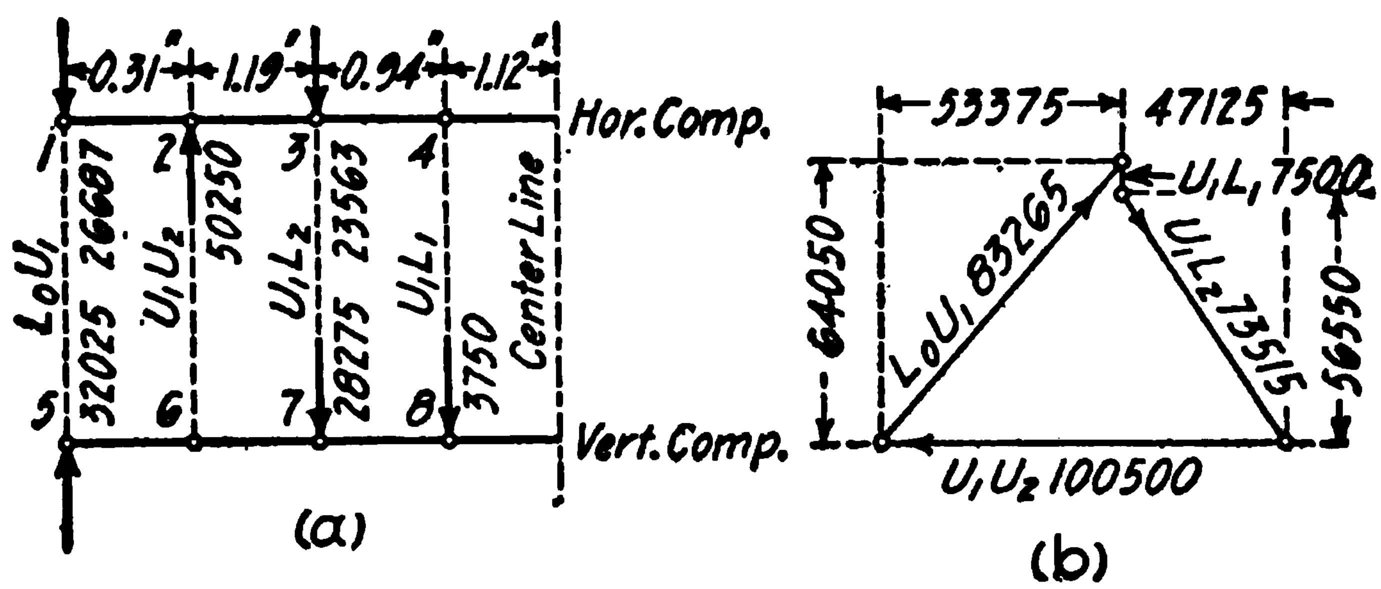


FIG. 294. PIN U_1. MAXIMUM STRESS IN U_1L_2.

(b) *Maximum Stress in Main Tie* U_1L_2.—The total stresses are given in (b) and the horizontal and vertical components are given in (a) Fig. 294.

Horizontal Components.—
Bending moments at 1 = 0.
Bending moments at 2 = 26,687 × 0.31″ = 8,270 in.-lbs.
Bending moments at 3 & 4 = 26,687 × 1.50″ − 50,250 × 1.19″ = 19,770 in.-lbs.

Vertical Components.—
Bending moment at 5 = 0.
Bending moment at 7 = 32,025 × 1.50″ = 48,038 in.-lbs.
Bending moment at 8 = 32,025 × 2.44″ − 28,275 × 0.94″ = 50,430 in.-lbs.

Maximum bending moment is at 4 and $8 = \sqrt{19{,}770^2 + 50{,}430^2}$
$= 54{,}000$ in.-lbs.
Efficiency $= 63{,}650/54{,}000 = 1.18$.

Bearing on Pin U_1. (1) *Bearing of End-post L_0U_1 on Pin U_1.*—The maximum bearing occurs with maximum stress in the end-post and the top chord.

Bearing area of 2 $\frac{7}{16}''$ pls. $= 2 \times \frac{7}{16}'' \times 3\frac{3}{16}''$	$= 2.79$ sq. in.
Bearing area of 2 $\frac{5}{16}''$ pls. $= 2 \times \frac{5}{16}'' \times 3\frac{3}{16}''$	$= 1.99$ sq. in.
Bearing area of 2 $\frac{1}{4}''$ pls. $= 2 \times \frac{1}{4}'' \times 3\frac{3}{16}''$	$= 1.59$ sq. in.
Bearing area of 2 [s 10″ @ 15 lbs., $= 2 \times 0.24'' \times 3\frac{3}{16}''$	$= 1.53$ sq. in.
Total bearing area	$= 7.90$ sq. in.

Maximum bearing stress $= 99{,}645$ lbs.
Actual unit bearing stress $= 99{,}645/7.90 = 12{,}620$ lbs.
Allowable unit bearing stress $= 18{,}000$ lbs.
Efficiency $= 18{,}000/12{,}620 = 1.42$.

(2) *Bearing of the Top Chord U_1U_2 on the Pin U_1.*—The maximum bearing stress occurs with a maximum stress in the top chord.

Bearing area of 2 $\frac{1}{4}''$ pls. $= 2 \times \frac{1}{4}'' \times 3\frac{3}{16}''$	$= 1.59$ sq. in.
Bearing area of 4 $\frac{3}{8}''$ pls. $= 4 \times \frac{3}{8}'' \times 3\frac{3}{16}''$	$= 4.78$ sq. in.
Bearing area of 2 [s 10″ @ 15 lbs. $= 2 \times 0.24'' \times 3\frac{3}{16}''$	$= 1.53$ sq. in.
Total bearing area	$= 7.90$ sq. in.

Maximum bearing stress $= 109{,}500$ lbs.
Actual unit bearing stress $= 109{,}500/7.90 = 13{,}900$ lbs.
Allowable unit bearing stress $= 18{,}000$ lbs.
Efficiency $= 18{,}000/13{,}900 = 1.29$.

(3) *Bearing of Hip Vertical U_1M_1 on Pin U_1.*—Bearing area of 2 $\frac{7}{8}''$ bars $= 2 \times \frac{7}{8}'' \times 3\frac{3}{16}'' = 5.58$ sq. in.
Actual unit bearing stress $= 21{,}900/5.58 = 3{,}920$ lbs.
Allowable unit bearing stress $= 4/3(15{,}100) = 20{,}130$ lbs. (Specifications, § 76).
Efficiency $= 20{,}130/3{,}920 = 5.10$.

The pin has a diameter greater than $\frac{3}{4}$ the depth of the bar and is safe (Specifications, § 76).

(4) *Bearing of Main Tie U_1L_2 on Pin U_1.*—Bearing area of 2 bars $2\frac{1}{2}'' \times \frac{7}{8}'' = 2 \times \frac{7}{8}'' \times 3\frac{3}{16}'' = 5.25$ sq. in.
Actual unit bearing stress $= 73{,}515/5.25 = 14{,}000$ lbs.
Allowable unit bearing stress $= 4/3(15{,}060) = 20{,}080$ lbs. (Specifications, § 76).

The diameter of the pin is greater than ¾ the depth of the bar (Specifications, § 76).

Shear on Pin U_1. (a) *Maximum Stress in L_0U_1 and U_1U_2. Horizontal Components.*—
Shear between 1 & 2 = 31,937 lbs.
Shear between 2 & 3 = 22,813 lbs.
Shear between 3 & 4 = 0.

Vertical Components.—
Shear between 5 & 7 = 38,325 lbs.
Shear between 7 & 8 = 10,950 lbs.
Shear between 8 & 8′ = 0.
Maximum shear occurs between 1 & 2 and 5 & 6 and

$$= \sqrt{31{,}937^2 + 38{,}325^2} = 49{,}823 \text{ lbs.}$$

Allowable shear = area $3\frac{3}{16}''$ pin × 10,000 = 7.979 × 10,000 = 79,790 lbs.

Efficiency = 79,790/49,823 = 1.60.

(b) *Maximum Stress in U_1L_2. Horizontal Components.*—
Shear between 1 & 2 = 26,687 lbs.
Shear between 2 & 3 = 23,563 lbs.
Shear between 3 & 4 = 0.

Vertical Components.—
Shear between 5 & 7 = 32,025 lbs.
Shear between 7 & 8 = 3,750 lbs.
Shear between 8 & 8′ = 0.
Maximum shear occurs between 1 & 2 and 5 & 6 and

$$= \sqrt{26{,}687^2 + 32{,}025^2} = 41{,}633 \text{ lbs.}$$

This is less than when there is a maximum stress in the end-post and top chord.

PIN L_1.—The pin is $2\frac{11}{16}''$ diameter; grip $12\frac{3}{4}''$; length $15\frac{1}{4}''$.

Bending Moment. (a) *Wind Not Considered.*—The maximum moments occur with maximum stresses in L_0L_1 and L_1L_2.

Horizontal Components.—
Bending moment at 1 = 0.
Bending moment at 2 = 31,940 × 0.94″ = 30,023 in.-lbs.
Bending moment at 3 = 0.

The vertical components are transmitted directly to the hip vertical and produce no bending in the pin.
Maximum bending moment in the pin = 30,023 in.-lbs.
Allowable bending moment (Table XXXIV) = 38,150 in.-lbs.
Efficiency for dead and live loads = 38,150/30,023 = 1.27.

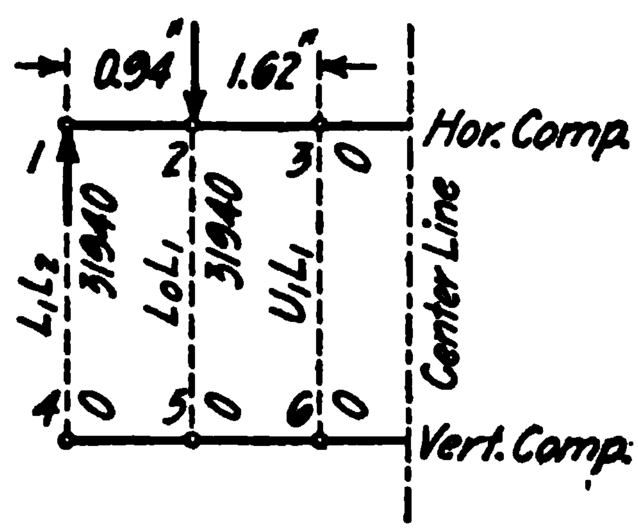


FIG. 295. PIN L_1.

(b) *Wind Considered.*—The total stresses in L_1 and L_2, including wind, = —42,000 — 21,875 — 41,285 = — 105,160 lbs., of which 52,580 lbs. comes on one bar. The wind stress is 53.8 per cent of the dead and live load stress and must be considered.
Maximum bending moment = 52,580 × 0.94 = 49,400 in.-lbs.
The allowable bending moment including wind moment = 38,150 × 1.25 = 47,690 in.-lbs.
Efficiency = 47,690/49,400 = 0.96.

Bearing on Pin L_1. (1) *Bearing of L_0L_1 and L_1L_2 on Pin L_1.* (a) *Wind Not Considered.*—Bearing area of 2 bars $2\frac{1}{2}'' \times \frac{7}{8}'' = 2 \times \frac{7}{8}'' \times 2\frac{11}{16}'' = 4.73$ sq. in.
Actual unit bearing stress = 63,875/4.73 = 13,500 lbs.
Allowable unit bearing stress = 4/3 × 15,060 = 20,080 lbs. (§ 76).
Efficiency = 20,080/13,500 = 1.50.

(b) *Wind Considered.*—
Actual unit bearing stress = 105,160/4.73 = 22,200 lbs.
Allowable unit bearing stress = 4/3 × 15,060 × 1.25 = 25,080 lbs.
Efficiency = 25,080/22,200 = 1.11.

The pin has a diameter greater than $\frac{3}{4}$ the depth of the bar (Specification § 76).

Shear on Pin L_1. (a) *Wind Not Considered.*—Maximum shear = 31,940 lbs.

Allowable shear on the pin (§ 40a) = 5.67 × 10,000 = 57,600 lbs.
Efficiency = 57,600/31,940 = 1.80.

(b) *Wind Considered.*—Maximum shear = 52,580 lbs.
Allowable shear considering wind (§ 46) = 57,600 × 1.25 = 72,000 lbs.
Efficiency = 72,000/52,580 = 1.37.

PIN M_1.—The pin is $2\frac{11}{16}''$ diameter; grip 7″; length 11″. There are no wind stresses nor horizontal components.

Bending Moments.—Maximum bending moment = 10,950 × 2.60″ = 28,470 in.-lbs.
Allowable bending moment (Table XXXIV) = 38,150 in.-lbs.
Efficiency = 38,150/28,470 = 1.34.

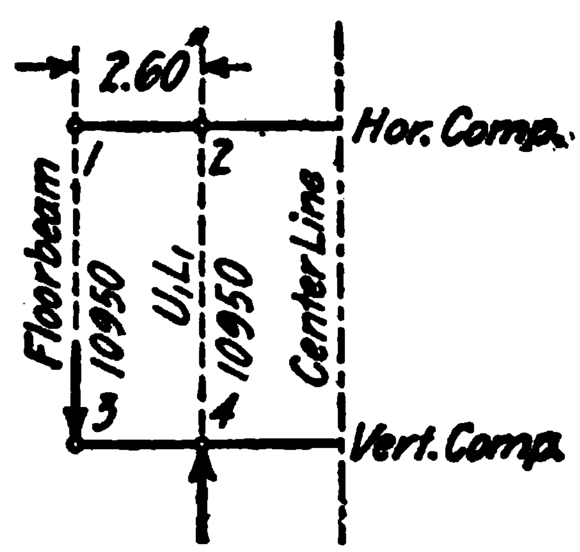


FIG. 296. PIN M_1.

Bearing on Pin M_1. (1) *Bearing on M_1L_1.*—

Bearing area of 2 $\frac{1}{4}''$ pls. = 2 × $\frac{1}{4}''$ × $2\frac{11}{16}''$	= 1.34 sq. in.
Bearing area of 2 [s 6″ @ 8 lbs. = 2 × 0.20″ × $2\frac{11}{16}''$	= 1.08 sq. in.
Total bearing area	= 2.42 sq. in.

Maximum unit bearing stress = 21,900/2.42 = 9,050 lbs.
Allowable unit bearing stress = 18,000 lbs.
Efficiency = 18,000/9,050 = 1.99.

(2) *Bearing on M_1U_1.*—Bearing area of 2 bars $\frac{7}{8}''$ square = 2 × $\frac{7}{8}''$ × $2\frac{11}{16}''$ = 4.70 sq. in.
Maximum unit bearing stress = 21,900/4.70 = 4,650 lbs.
Allowable unit bearing stress = 4/3(15,100) = 20,130 lbs. (§ 76).
Efficiency = 20,130/4,650 = 4.30.

Shear on Pin M_1.—
Maximum shear = 10,950 lbs.
Allowable shear = 57,600 lbs.
Efficiency = 57,600/10,950 = 5.25.

PIN L_2.—The pin is $3\frac{3}{16}''$ diameter; grip $13\frac{1}{8}''$; length $15\frac{5}{8}''$.

Bending Moment. (1) *Maximum Stress in Lower Chords L_1L_2 and L_2L_3.* (a) *Wind Not Considered.*

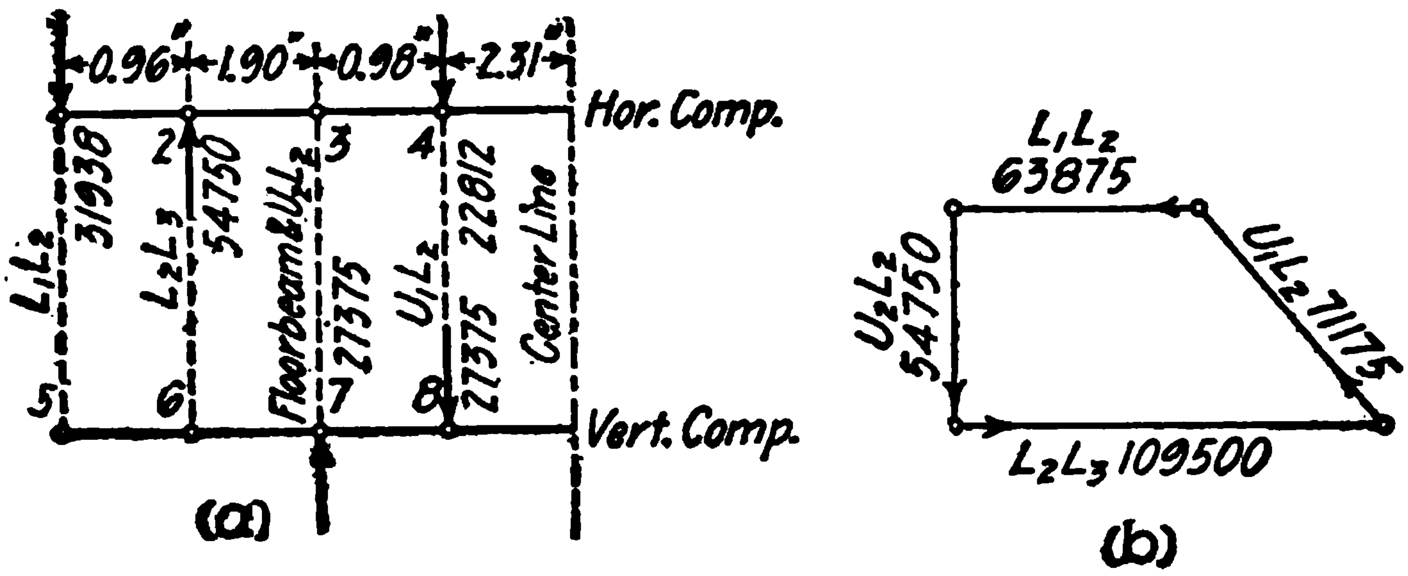


FIG. 297. PIN L_2. MAXIMUM STRESSES IN LOWER CHORDS.

Horizontal Components.—

Bending moment at $2 = 31,938 \times 0.96'' = 30,660$ in.-lbs.

Bending moment at $3 = 31,938 \times 2.86'' - 54,750 \times 1.90'' = 12,680$ in.-lbs.

Bending moment at $4 = 31,938 \times 3.84'' - 54,750 \times 2.88'' = 35,000$ in.-lbs.

Vertical Components.—

Bending moment at $7 = 0$.

Bending moment at $8 = 27,375 \times 0.98'' = 26,830$ in.-lbs.

Maximum bending moment is at 4 and 8 and $= \sqrt{35,000^2 + 26,830^2} = 44,000$ in.-lbs.

Allowable bending moment on a $3\frac{3}{16}''$ pin (Table XXXIV) $= 63,650$ in.-lbs.

Efficiency $= 63,650/44,000 = 1.44$.

(b) *Wind Considered.*—The maximum stress in L_1L_2 due to wind + dead + live load $= 105,160 \times \frac{1}{2} = 52,580$ lbs. The maximum stress in $L_2L_3 = 177,260 \times \frac{1}{2} = 88,630$ lbs.

Horizontal Components.—

Bending moment at $2 = 52,580 \times 0.96'' = 50,500$ in.-lbs.

Bending moment at $4 = 52,580 \times 3.84'' - 88,630 \times 2.88'' = 52,000$ in.-lbs.

Vertical Components.—

Bending moment at $7 = 0$.

Bending moment at 8 = 27,375 × 0.98″ = 26,830 in.-lbs.
Maximum bending moment occurs at points 4 and 8 and

$$= \sqrt{52{,}000^2 + 26{,}830^2} = 59{,}000 \text{ in.-lbs.}$$

Allowable bending moment considering wind = 63,650 × 1.25 = 79,450 in.-lbs.

Efficiency = 79,450/59,000 = 1.35.

(2) *Maximum Stress in Main Tie U_1L_2.*

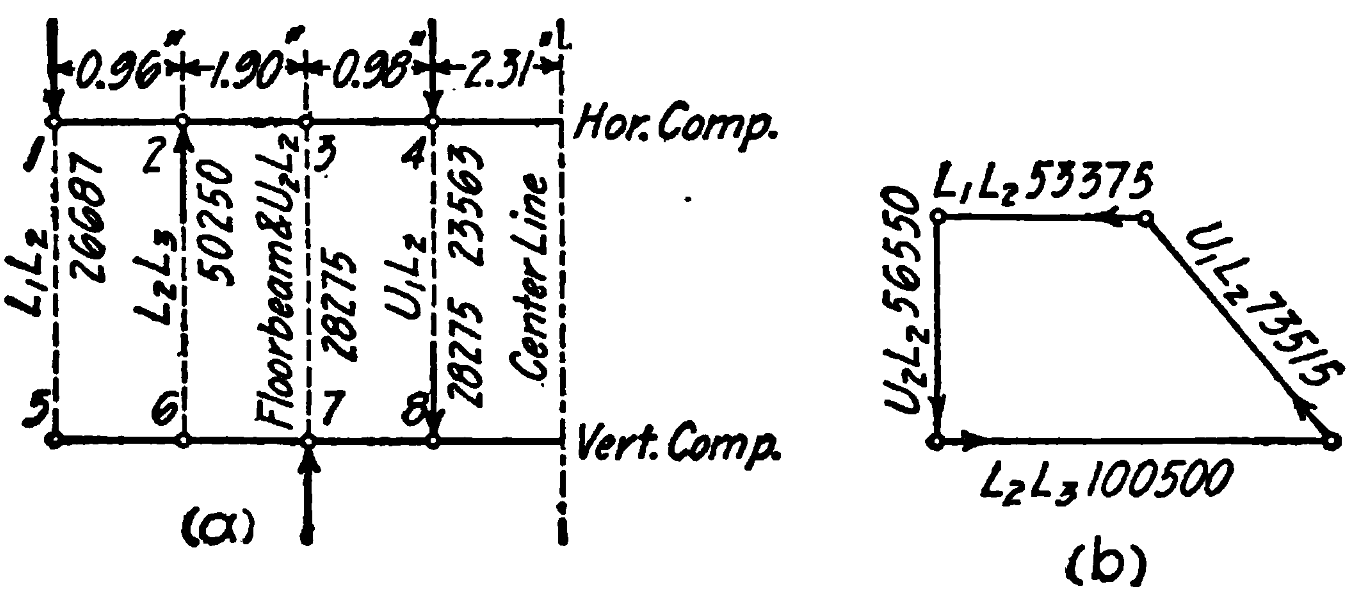


FIG. 298. PIN L_2. MAXIMUM STRESS IN MAIN TIE U_1L_2.

Horizontal Components.—
Bending moment at 2 = 26,687 × 0.96″ = 25,620 in.-lbs.
Bending moment at 3 = 26,687 × 2.86″ — 50,250 × 1.90″ = 19,150 in.-lbs.
Bending moment at 4 = 26,687 × 3.84″ — 50,250 × 2.88″ = 46,250 in.-lbs.

Vertical Components.—
Bending moment at 7 = 0.
Bending moment at 8 = 28,275 × 0.98″ = 27,700 in.-lbs.
Maximum bending moment occurs at 4 and 8 and

$$= \sqrt{46{,}250^2 + 27{,}700^2} = 54{,}000 \text{ in.-lbs.}$$

Allowable bending moment = 63,650 in.-lbs.
Efficiency = 63,650/54,000 = 1.18.

Bearing on Pin L_2. (1) *Bearing of Intermediate Post U_2L_2 on Pin L_2.*

Bearing area of 2 $\frac{3}{8}$″ pls. = 2 × $\frac{3}{8}$″ × $3\frac{3}{16}$″	= 2.39 sq. in.
Bearing area of 2 [s 7″ @ $9\frac{3}{4}$ lbs. = 2 × 0.21″ × $3\frac{3}{16}$″	= 1.34 sq. in.
Total bearing area	= 3.73 sq. in.

Maximum stress $= 56{,}550$ lbs.
Maximum unit bearing stress $= 56{,}550/3.73 = 15{,}200$ lbs.
Allowable unit bearing stress $= 18{,}000$ lbs.
Efficiency $= 18{,}000/15{,}200 = 1.18$.

(2) *Bearing of Main Tie U_1L_2.*—Bearing area of 2 bars $2\frac{1}{2}'' \times \frac{7}{8}'' = 2 \times \frac{7}{8}'' \times 3\frac{1}{16}'' = 5.38$ sq. in.
Actual unit bearing stress $= 73{,}515/5.38 = 13{,}600$ lbs.
Allowable unit bearing stress $= 4/3(15{,}000) = 20{,}000$ lbs. (§ 76).
Efficiency $= 20{,}000/13{,}600 = 1.47$.

The diameter of the pin is greater than $\frac{3}{4}$ of the depth of the bar (§ 76).

(3) *Bearing of the Lower Chord L_1L_2.*—Bearing area of 2 bars $2\frac{1}{2}'' \times \frac{7}{8}'' = 2 \times \frac{7}{8}'' \times 3\frac{1}{16}'' = 5.38$ sq. in.

(a) *Wind Not Considered.*—
Actual unit bearing stress $= 63{,}875/5.38 = 11{,}900$ lbs.
Allowable unit bearing stress $= 4/3(15{,}060) = 20{,}080$ lbs. (§ 76).
Efficiency $= 20{,}080/11{,}900 = 1.70$.

(b) *Wind Considered.*—
Actual unit bearing stress $= 105{,}160/5.38 = 19{,}500$ lbs.
Allowable unit bearing stress when wind is considered $= 20{,}080 \times 1.25 = 25{,}100$ in.-lbs.
Efficiency $= 25{,}100/19{,}500 = 1.30$.

(4) *Bearing of Lower Chord L_2L_3.*—Bearing area of 2 bars $3\frac{1}{2}'' \times \frac{15}{16}'' = 2 \times \frac{15}{16}'' \times 3\frac{3}{16}'' = 5.98$ sq. in.

(a) *Wind Not Considered.*—
Actual unit bearing stress $= 109{,}500/5.98 = 18{,}300$ lbs.
Allowable unit bearing stress $= 4/3(15{,}060) = 20{,}080$ lbs. (§ 76).
Efficiency $= 20{,}080/18{,}300 = 1.09$.

The diameter of the pin is greater than $\frac{3}{4}$ of the depth of the bar (Specifications, § 76).

(b) *Wind Considered.*—
Actual unit bearing stress when wind is considered $= 177{,}260/5.98 = 29{,}600$ lbs.
Allowable unit bearing stress when wind is considered $= 20{,}080 \times 1.25 = 25{,}100$ lbs.
Efficiency $= 25{,}100/29{,}600 = 0.85$.

Shear in Pin L_2. (a) *Wind Not Considered.* (1) *Maximum Stress in Lower Chords L_1L_2 and L_2L_3.*

Horizontal Components.—

Shear between 1 & 2 = 31,938 lbs.
Shear between 2 & 3 = 22,812 lbs.
Shear between 3 & 4 = 22,812 lbs.

Vertical Components.—

Shear between 7 & 8 = 27,375 lbs.
The maximum shear is between 3 & 4 and 7 & 8 and

$$= \sqrt{22,812^2 + 27,375^2} = 35,588 \text{ lbs.}$$

Allowable shear in a $3\frac{3}{16}''$ pin = 79,790 lbs.
Efficiency = 79,790/35,588 = 2.24.

(2) *Maximum Stress in Main Tie U_1L_2.*

Horizontal Components.—

Shear between 1 & 2 = 26,687 lbs.
Shear between 2 & 3 = 23,563 lbs.
Shear between 3 & 4 = 23,563 lbs.

Vertical Components.—

Shear between 7 & 8 = 28,275 lbs.
The maximum shear occurs between 3 & 4 and 7 & 8 and

$$= \sqrt{23,563^2 + 28,275^2} = 36,758 \text{ lbs.}$$

Allowable unit shear on a $3\frac{3}{16}''$ pin = 79,790 lbs.
Efficiency = 79,790/36,758 = 2.17.

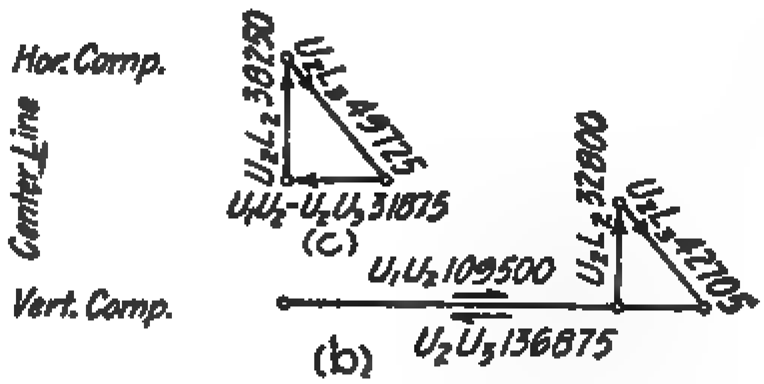


FIG. 299. PIN U_2.

PIN U_2.—The pin is $2\frac{11}{16}''$ diameter; grip 8″; length $10\frac{1}{8}''$. The maximum chord stresses at the joint are given in (b) Fig. 299, while the maximum web stresses at the joint are given in (c).

Bending Moment.—The maximum bending moment will occur with a maximum stress in U_2L_3 as given in (c) and in brackets in (a) Fig. 299.

Horizontal Components.—
Bending moment at $3 = 15{,}940 \times 1.38'' = 22{,}000$ in.-lbs.

Vertical Components.—
Bending moment at $6 = 19{,}125 \times 0.75'' = 14{,}340$ in.-lbs.
Maximum bending moment occurs at 3 and 6 and

$$= \sqrt{\overline{22{,}000}^2 + \overline{14{,}340}^2} = 26{,}260 \text{ in.-lbs.}$$

Allowable bending moment $= 38{,}150$ in.-lbs. (Table XXXIV).
Efficiency $= 38{,}150/26{,}260 = 1.44$.

Bearing on Pin U_2. (1) *Bearing of Upper Chord U_1U_2 and U_2U_3 on Pin U_2.*—

Bearing area of 2 $\frac{1}{4}''$ pls. $= 2 \times \frac{1}{4}'' \times 2\frac{11}{16}''$	$= 1.34$ sq. in.
Bearing area of 2 [s 10″ @ 15 lbs. $= 2 \times 0.24'' \times 2\frac{11}{16}''$	$= 1.29$ sq. in.
Total bearing area	$= 2.63$ sq. in.

Bearing stress in U_1U_2 and $U_2U_3 = 31{,}875$ lbs. [(c) Fig. 299].
Actual unit bearing stress $= 31{,}875/2.63 = 12{,}110$ lbs.
Allowable unit bearing stress $= 18{,}000$ lbs.
Efficiency $= 18{,}000/12{,}110 = 1.49$.

(2) *Bearing of Post U_2L_2 on Pin U_2.*—
Bearing area of 2 $\frac{1}{2}''$ pls. $= 2 \times \frac{1}{2}'' \times 2\frac{11}{16}'' = 2.69$ sq. in.
Actual unit bearing stress $= 38{,}250/2.69 = 14{,}230$ lbs.
Allowable unit bearing stress $= 18{,}000$ lbs.
Efficiency $= 18{,}000/14{,}230 = 1.26$.

(3) *Bearing of Main Tie U_2L_3 on Pin U_2.*—The diameter of the pin is greater than $\frac{2}{3}$ the depth of bar (Specifications, § 76).

Shear on Pin U_2.
Horizontal Components.—

Shear between 1 & $2 = 15{,}940$ lbs.
Shear between 2 & $3 = 15{,}940$ lbs.

Vertical Components.—

Shear between 5 & $6 = 19{,}125$ lbs.
Maximum shear occurs between 2 & 3 and 5 & 6 and

$$= \sqrt{15{,}940^2 + 19{,}125^2} = 24{,}863 \text{ lbs.}$$

Allowable shear on a $2\frac{11}{16}''$ pin $= 5.76 \times 10{,}000 = 57{,}600$ lbs.
Efficiency $= 57{,}600/24{,}863 = 2.30$.

PIN L_3.—The pin is $3\frac{5}{16}''$ diameter; grip $14\frac{3}{4}''$ length $17\frac{1}{4}''$.

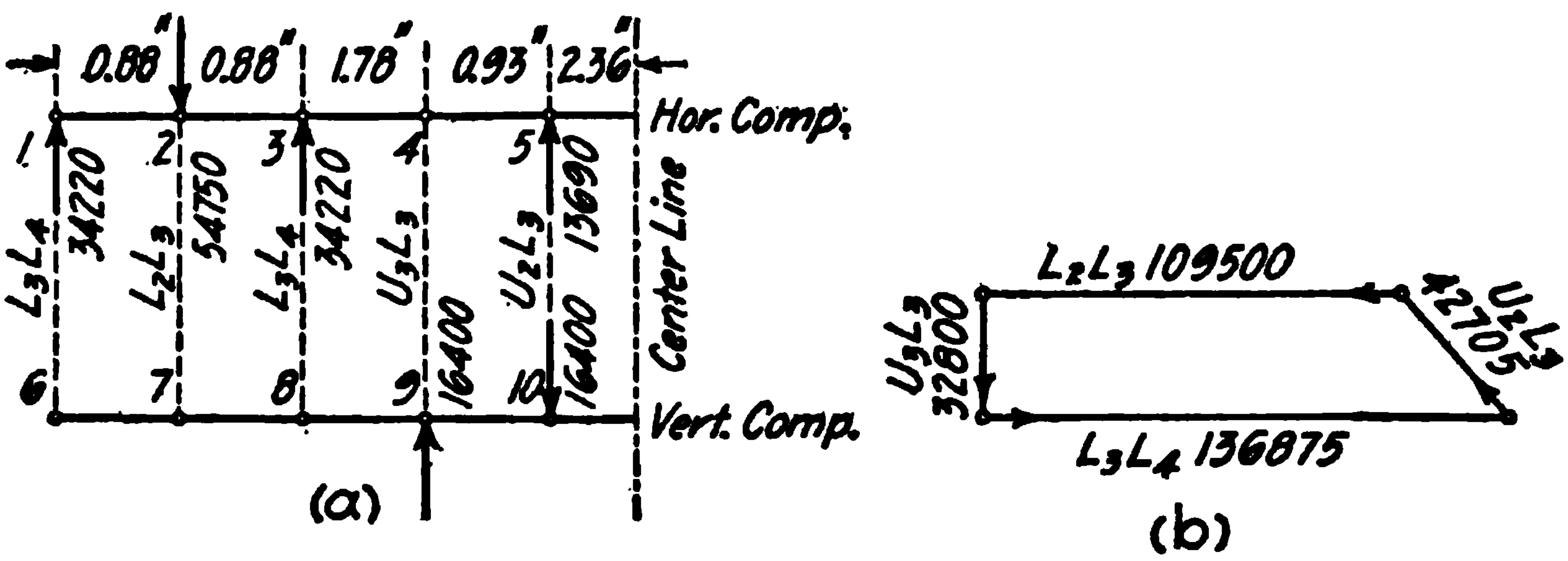


FIG. 300. PIN L_4. MAXIMUM STRESSES IN LOWER CHORDS.

Bending Moment.—(a) *Maximum Stresses in the Bottom Chords L_2L_3 and L_3L_4*, Fig. 300.

Horizontal Components.—

Bending moment at 2 = 34,220 × 0.88″ = 30,100 in.-lbs.

Bending moment at 3 = 34,220 × 1.76″ − 54,750 × 0.88″ = 12,050 in.-lbs.

Bending moment at 5 = 34,220 × 4.47″ − 54,750 × 3.59″ + 34,220 × 2.71″ = 49,150 in.-lbs.

Vertical Components.—

Bending moment at 10 = 16,400 × 0.93″ = 15,250 in.-lbs.

Maximum bending moment occurs at 5 and 10 and

$$= \sqrt{49,150^2 + 15,250^2} = 56,900 \text{ in.-lbs.}$$

Allowable bending moment = 63,650 in.-lbs.

Efficiency = 63,650/56,900 = 1.12.

(b) *Maximum Stress in Main Tie U_2L_3*, Fig. 301.

Horizontal Components.—

Bending moment at 2 = 28,595 × 0.88″ = 25,160 in.-lbs.

Bending moment at 3 = 28,595 × 1.76″ − 41,250 × 0.88″ = 14,000 in.-lbs.

Bending moment at 5 = 28,595 × 4.47″ − 41,250 × 3.59″ + 28,595 × 2.71″ = 56,900 in.-lbs.

Vertical Components.—

Bending moment at 10 = 19,125 × 0.93″ = 17,800 in.-lbs.

Maximum bending occurs at 5 and 8 and $= \sqrt{56,900^2 + 17,800^2}$ = 59,700 in.-lbs.

Efficiency = 63,560/59,700 = 1.06.

(c) *Wind Stresses Considered,* Fig. 300.—Total stress in L_2L_3 $= \frac{1}{2}(177{,}260) = 88{,}630$ lbs.; total stress in $L_3L_4 = \frac{1}{2}(220{,}540) = 110{,}270$ pounds.

Horizontal Components.—

Bending moment at $5 = 55{,}135 \times 4.47'' - 88{,}630 \times 3.59'' + 55{,}135 \times 2.71'' = 77{,}700$ in.-lbs.

Vertical Components.—

Bending moment at $10 = 18{,}040 \times 0.93'' = 16{,}780$ in.-lbs.

Maximum bending moment occurs at 5 and 10 and

$$= \sqrt{77{,}700^2 + 16{,}780^2} = 78{,}800 \text{ in.-lbs.}$$

Allowable bending moment when wind is considered $= 63{,}650 \times 1.25 = 79{,}570$ in.-lbs.

Efficiency $= 79{,}570/78{,}800 = 1.01$.

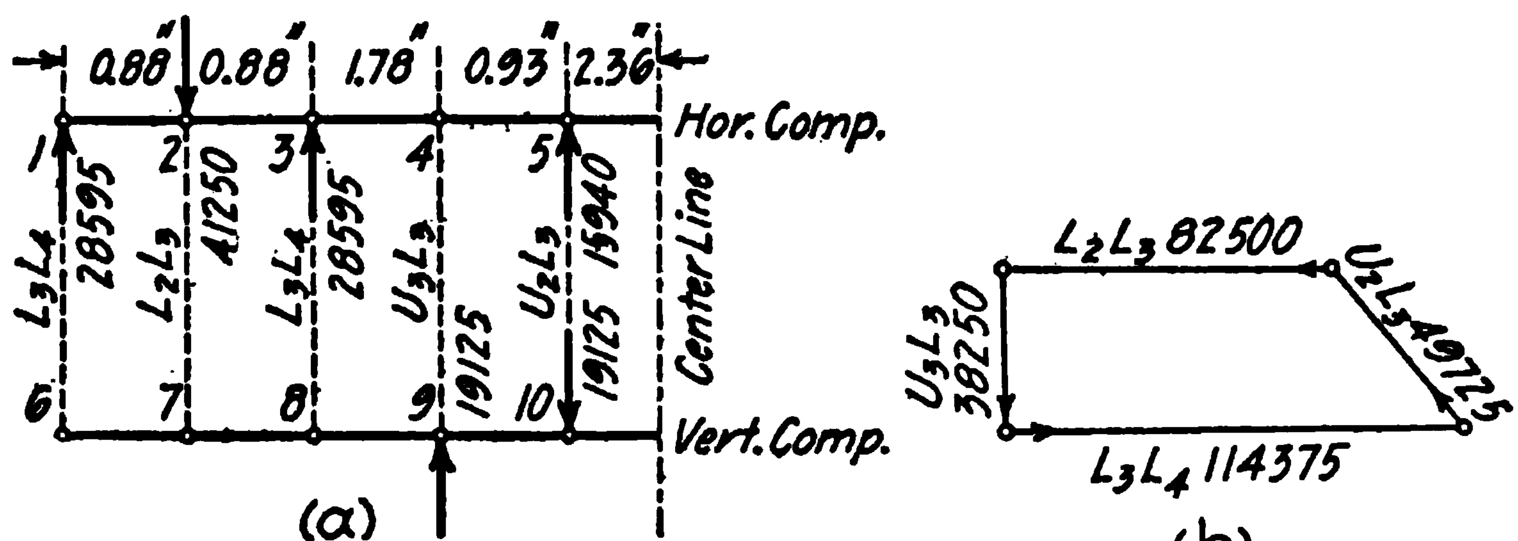


FIG. 301. PIN L_3. MAXIMUM STRESS IN MAIN TIE U_2L_3.

Bearing on Pin L_3. (1) *Bearing of Intermediate Post U_3L_3 on Pin L_3.*—

Bearing area of 2 $\frac{3}{8}''$ pls. $= 2 \times \frac{3}{8}'' \times 3\frac{3}{16}''$ $= 2.39$ sq. in.

Bearing area of 2 [s 7″ @ $9\frac{3}{4}$ lbs. $= 2 \times 0.21'' \times 3\frac{3}{16}'' = 1.34$ sq. in.

Total bearing area $= 3.73$ sq. in.

Maximum stress in U_3L_3 bearing on the pin $= 38{,}250$ lbs., Fig. 301.

Actual unit bearing stress $= 38{,}250/3.73 = 10{,}400$ lbs.

Allowable unit bearing stress $= 18{,}000$ lbs.

Efficiency $= 18{,}000/10{,}400 = 1.73$.

(2) *Bearing of Lower Chord L_2L_3 on Pin L_3.*—The diameter of the pin is greater than $\frac{2}{3}$ of the depth of the bar (Specifications, § 76).

(3) *Bearing of Lower Chord L_3L_4 on Pin L_3.*—The diameter of the pin is greater than $\frac{2}{3}$ the depth of the bar (Specifications, § 76).

(4) *Bearing of Main Tie U_2L_3 on Pin L_3.*—The diameter of the pin is greater than $\frac{3}{4}$ of the depth of the bar (Specifications, § 76).

Shear on Pin L_3. (a) *Maximum Stresses in the Lower Chords* L_2L_3 and L_3L_4, Fig. 300.

Horizontal Components.—
Shear between 1 & 2 = 34,220 lbs.
Shear between 2 & 3 = 20,530 lbs.
Shear between 3 & 4 = 13,690 lbs.
Shear between 4 & 5 = 13,690 lbs.

Vertical Components.—
Shear between 6 & 9 = 0.
Shear between 9 & 10 = 16,400 lbs.

The maximum shear comes between 1 & 2 and = 34,220 lbs.
Allowable shear on a $3\frac{3}{16}''$ pin $= 7.979 \times 10{,}000 = 79{,}790$ lbs.
Efficiency $= 79{,}790/34{,}220 = 2.33$.

(b) *Maximum Stress in Main Tie U_2L_3*, Fig. 301.
Horizontal Components.—
Shear between 1 & 2 = 28,595 lbs.
Shear between 2 & 3 = 12,655 lbs.
Shear between 3 & 4 = 15,940 lbs.

Vertical Components.—
Shear between 6 & 9 = 0.
Shear between 9 & 10 = 19,125 lbs.
The maximum shear comes between 1 & 2 and = 28,595 lbs.
This shear is less than for case (a) above.

PIN L_4.—The pin is $3\frac{3}{16}''$ diameter; grip 16″; length $18\frac{1}{2}''$.

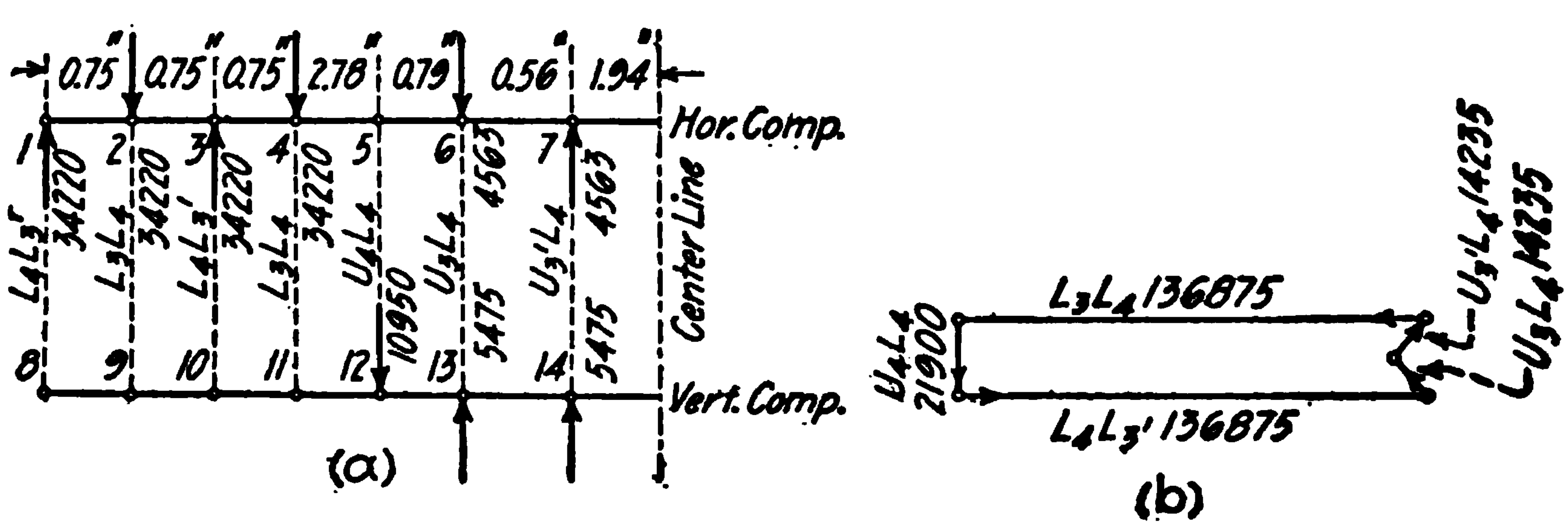


FIG. 302. PIN L_4. MAXIMUM STRESSES IN LOWER CHORDS.

Note.—Stress given as U_4L_4 is floorbeam reaction.

Bending Moment. (a) *Maximum Stress in Lower Chord* L_3L_4, Fig. 302.

Horizontal Components.—

Bending moment at 1 = 0.

Bending moment at 2 = 34,220 × 0.75″ = 25,665 in.-lbs.

Bending moment at 3 = 34,220 × 1.50″ — 34,220 × 0.75″ = 25,665 in.-lbs.

Bending moment at 4 = 34,220 × 2.25″ — 34,220 × 1.50″ + 34,220 × 0.75″ = 51,330 in.-lbs.

Bending moment at 5 = 34,220 × 5.03″ — 34,220 × 4.28″ + 34,220 × 3.53″ — 34,220 × 2.78″ = 51,330 in.-lbs.

Bending moment at 6 = 34,220 × 5.82″ — 34,220 × 5.07″ + 34,220 × 4.32″ — 34,220 × 3.57″ = 51,330 in.-lbs.

Bending moment at 7 = 34,220 × 6.38″ — 34,220 × 5.63″ + 34,220 × 4.85″ — 34,220 × 4.13″ — 4,563 × 0.56″ = 48,775 in.-lbs.

Vertical Components.—

Bending moment at 12 = 0.

Bending moment at 13 = 10,950 × 0.79″ = 8,650 in.-lbs.

Bending moment at 14 = 10,950 × 1.35″ — 5,475 × 0.56″ = 11,717 in.-lbs.

The maximum bending moment occurs at 6 and 13 and

$$= \sqrt{51,330^2 + 8,650^2} = 58,000 \text{ in.-lbs.}$$

Allowable bending moment = 63,650 in.-lbs.

Efficiency = 63,650/58,000 = 1.10.

(b) *Maximum Stress in Main Tie* U_3L_4, Fig. 303. By comparing with case (a) it is seen that the maximum bending occurs with that condition.

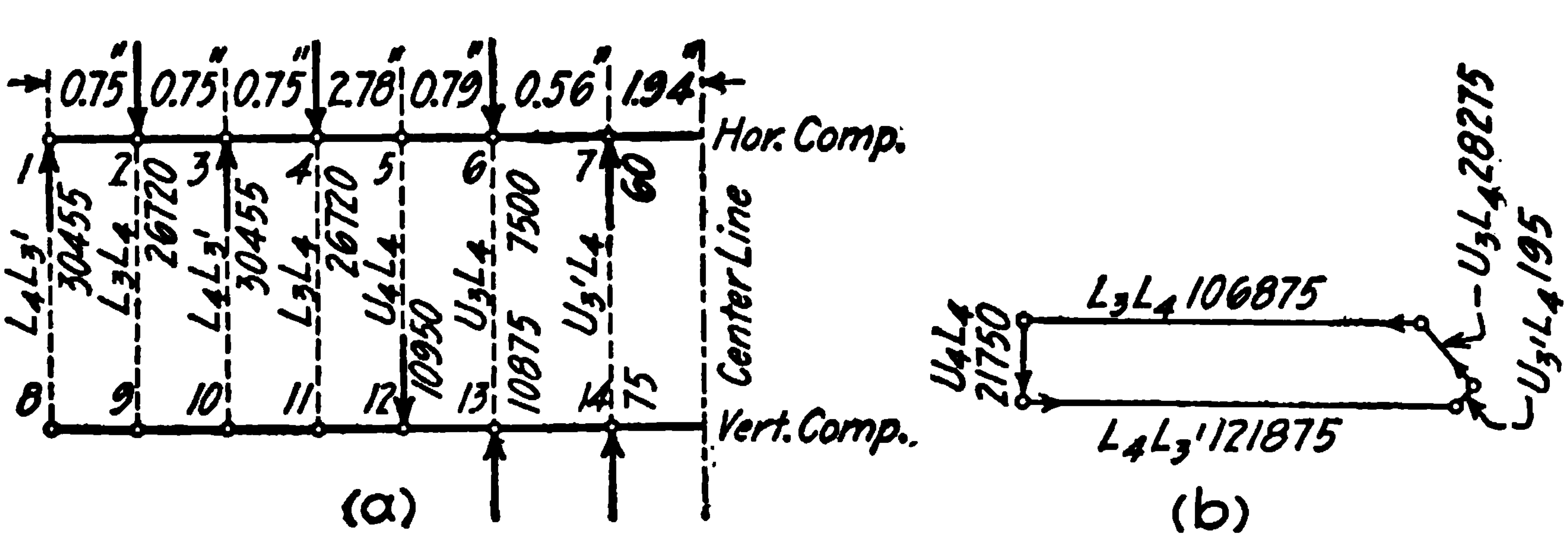


FIG. 303. PIN L_4. MAXIMUM STRESS IN MAIN TIE U_3L_4.

(c) *Considering Wind.*—The maximum stresses in $L_2L_4 = 220{,}540$ pounds.

Horizontal Components.—

The maximum bending moment occurs at 4 and $= 55{,}135 \times 2.25'' - 55{,}135 \times 1.50'' + 55{,}135 \times 0.75'' = 82{,}700$ in.-lbs.

Allowable bending moment when wind is considered $= 63{,}650 \times 1.25 = 79{,}570$ in.-lbs.

Efficiency $= 79{,}570/82{,}700 = 0.96$.

Bearing on Pin L_4. (1) *Bearing of Intermediate Post U_4L_4*, Fig. 303.

Bearing area of 2 $\frac{3}{8}''$ pls. $= 2 \times \frac{3}{8}'' \times 3\frac{3}{16}''$ $= 2.39$ sq. in.

Bearing area of 2 [s 6″ @ 8 lbs. $= 2 \times 0.20'' \times 3\frac{3}{16}'' = 1.27$ sq. in.

Total bearing area $= 3.66$ sq. in.

Actual unit bearing stress $= 21{,}900/3.66 = 6{,}000$ lbs.

Allowable unit bearing stress $= 18{,}000$ lbs.

Efficiency $= 18{,}000/6{,}000 = 3.00$.

Shear on Pin L_4.

Horizontal Components.—

Shear between 1 & 2 $= 34{,}220$ lbs.

Shear between 2 & 3 $= 0$.

Shear between 3 & 4 $= 34{,}220$ lbs.

Shear between 4 & 5 $= 0$.

Shear between 5 & 6 $= 0$.

Shear between 6 & 7 $= 4{,}563$ lbs.

Vertical Components.—

Shear between 12 & 13 $= 10{,}950$ lbs.

Shear between 13 & 14 $= 5{,}475$ lbs.

The maximum shear is between 1 & 2 and 3 & 4 and $= 34{,}220$ lbs.

The allowable shear on a $3\frac{3}{16}''$ pin $= 79{,}790$ lbs.

Efficiency $= 79{,}790/34{,}220 = 2.33$.

SHEAR ON RIVETS. PIN L_0. (1) *Member L_0U_1*, Fig. 291.

Thickness of web $= 0.240$ in.

Thickness of outside plate $= 0.375$ in.

Thickness of inside plate $= 0.25$ in.

Total thickness of bearing $= 0.865$ in.

The total stress in $U_1L_0 = +65{,}520 + 34{,}125 = 99{,}645$ lbs.

The stress carried by the metal on one side $= \frac{1}{2}(99{,}645) = 49{,}823$ lbs.

The direct stress due to wind $= 12{,}500 + 7{,}000 = 19{,}500$ lbs. The direct stress due to wind is less than 25 per cent of the allowable stress for dead and live loads, and may be neglected (§ 46).

Stress in outside plate $= (49{,}823 \times 0.375)/0.865 = 21{,}650$ lbs.

16 $\frac{3}{4}''$ rivets transmit the stress in single shear.

Actual shear on one rivet $= 21{,}650/16 = 1{,}360$ lbs.

Allowable shear on one rivet in single shear $= 4{,}418$ lbs. (Table LXXVII).

Stress in inside plate $= (49{,}823 \times 0.25)/0.865 = 14{,}430$ lbs.

TABLE LXXVI.

ALLOWABLE BEARING AND SHEARING VALUES FOR $\frac{5}{8}''$ RIVETS IN HIGHWAY BRIDGES

Thickness of Plate	Truss				Floor System				Lateral System			
	Bearing		Shear		Bearing		Shear		Bearing		Shear	
	Shop	Field	Shop	Field	Shop	Field	Shop	Field	Shop	Field	Shop	Field
1/4	2813	1875	3068	2045	2250	1500	2454	1636	3938	2625	4295	2863
5/16	3515	2343	"	"	2812	1875	"	"	4921	3281	"	"
3/8	4219	2813	"	"	3375	2250	"	"	5907	3937	"	"
7/16	4922	3281	"	"	3938	2625	"	"	6891	4593	"	"
1/2	5625	3750	"	"	4500	3000	"	"	7875	5250	"	"
9/16	6328	4219	"	"	5062	3375	"	"	8660	5906	"	"
5/8	7031	4688	"	"	5625	3750	"	"	9843	6562	"	"

Allowable unit bearing stress 18000; and unit shearing stress 10000 lbs. per sq. in. For field rivets use $\frac{2}{3}$; for floor systems use 80 per cent; and for laterals use 140 per cent of these values.

TABLE LXXVII.

ALLOWABLE BEARING AND SHEARING VALUES FOR $\frac{3}{4}''$ RIVETS IN HIGHWAY BRIDGES

Thickness of Plate	Truss				Floor System				Lateral System			
	Bearing		Shear		Bearing		Shear		Bearing		Shear	
	Shop	Field	Shop	Field	Shop	Field	Shop	Field	Shop	Field	Shop	Field
1/4	3375	2250	4418	2945	2700	1800	3534	2356	4725	3150	6185	4123
5/16	4219	2813	"	"	3375	2250	"	"	5906	3937	"	"
3/8	5062	3375	"	"	4050	2700	"	"	7087	4725	"	"
7/16	5906	3938	"	"	4725	3150	"	"	8268	5512	"	"
1/2	6750	4500	"	"	5400	3600	"	"	9450	6300	"	"
9/16	7594	5062	"	"	6075	4050	"	"	10630	7088	"	"
5/8	8438	5625	"	"	6750	4500	"	"	11810	7875	"	"
11/16	9282	6138	"	"	7425	4950	"	"	13000	8662	"	"
3/4	10125	6750	"	"	8100	5400	"	"	14175	9450	"	"

Allowable unit bearing stress 18000; and unit shearing stress 10000 lbs. per sq. in. For field rivets use $\frac{2}{3}$; for floor systems use 80 per cent; and for laterals use 140 per cent of these values.

7 ¾″ rivets transmit the stress in single shear.
Actual shear on one rivet = 14,430/7 = 2,060 lbs.
Allowable shear on one rivet in single shear = 4,418 lbs.

Pin U_1, Fig. 293. (1) *Member* L_0U_1.—

Thickness of web	= 0.24	in.
Thickness of first outside plate	= 0.25	in.
Thickness of second outside plate	= 0.4375	in.
Thickness of inside plate	= 0.3125	in.
Total thickness of bearing	= 1.24	in.

The stress on one side of the member = 49,823 lbs.
Stress in first outside plate = (49,823 × 0.25)/1.24 = 10,060 lbs.
9 ¾″ rivets transmit the stress in single shear.
Actual shear on one rivet = 10,060/9 = 1,120 lbs.
Allowable shear on one rivet in single shear = 4,418 lbs.
Stress in second outside plate = (49,823 × 0.6875)/1.24 = 27,680 lbs.
15 ¾″ rivets transmit the stress in single shear.
Actual shear on one rivet = 27,680/15 = 1,846 lbs.
Allowable shear on one rivet = 4,418 lbs.
Stress in inside plate = (49,823 × 0.3125)/1.24 = 12,590 lbs.
9 ¾″ rivets transmit the stress in single shear.
Actual shear on one rivet = 12,590/9 = 1,400 lbs.
Allowable shear on one rivet = 4,418 lbs.

Rivets are safe for shear.

Member U_1U_2.—

Thickness of web	= 0.24	in.
Thickness of outside plate	= 0.375	in.
Thickness of first inside plate	= 0.25	in.
Thickness of second inside plate	= 0.375	in.
Total thickness of bearing	= 1.24	in.

The stress on one side of the member = ½(109,500) = 54,750 lbs.
Stress on outside plate = (54,750 × 0.375)/1.24 = 16,610 lbs.
18 ¾″ rivets transmit the stress in single shear.
Actual shear on one rivet = 16,610/18 = 920 lbs.
Allowable shear on one rivet = 4,418 lbs.
Stress in first inside plate = (54,750 × 0.25)/1.24 = 11,080 lbs.
9 ¾″ rivets transmit the stress in single shear.
Actual shear on one rivet = 11,080/9 = 1,230 lbs.
Allowable shear on one rivet = 4,418 lbs.
Stress in second inside plate = (54,750 × 0.625)/1.24 = 27,690 lbs.

9 $\frac{3}{4}''$ rivets transmit the stress in single shear.
Actual shear on one rivet = 27,600/9 = 3,077 lbs.
Allowable shear on one rivet = 4,418 lbs.
Rivets are all safe for shear.

Pin U_2, Fig. 299. (1) *Upper Chords U_1U_2 and U_2U_3.*—

Thickness of web = 0.24 in.
Thickness of plate = 0.25 in.
Total thickness of bearing = 0.49 in.

The maximum stress transmitted through the pin is the horizontal component of U_2L_3 = 31,850 lbs. The stress on one side = 15,925 lbs.

Stress in the plate = (15,925 × 0.25)/0.49 = 8,130 lbs.
12 $\frac{3}{4}''$ rivets transmit the stress in single shear.
Actual shear on one rivet = 8,130/12 = 680 lbs.
Allowable shear on one rivet = 4,418 lbs.

Intermediate Post U_2L_2.—

Thickness of web = 0.21 in.
Thickness of plate = 0.50 in.
Total thickness of bearing = 0.71 in.

Stress in plate = (19,125 × 0.50)/0.71 = 13,480 lbs.
9 $\frac{3}{4}''$ rivets transmit the stress in single shear.
Actual shear on one rivet = 13,480/9 = 1,500 lbs.
Allowable shear on one rivet = 4,418 lbs.

Pin U_3.—The upper chord splice is investigated on page 508.

Intermediate Post U_3L_3.—The maximum stress in U_3L_4 = 21,750 lbs.

Thickness of web = 0.21 in.
Thickness of plate = 0.375 in.
Total thickness of bearing = 0.585 in.

Stress in plate = (10,875 × 0.375)/0.585 = 6,980 lbs.
6 $\frac{3}{4}''$ rivets transmit the stress in single shear.
Actual shear in one rivet = 6,980/6 = 1,163 lbs.
Allowable shear on one rivet = 4,418 lbs.

Pin U_4.—Rivets safe for shear.

Pin M_1.—Fig. 296.

Thickness of web = 0.20 in.
Thickness of plate = 0.25 in.
Total thickness of bearing = 0.45 in.

Stress in the plate $= (10{,}950 \times 0.25)/0.45 = 6{,}083$ lbs.
4 ¾″ rivets transmit the stress in single shear.
Actual shear on one rivet $= 6{,}083/4 = 1{,}521$ lbs.
Allowable shear on one rivet $= 4{,}418$ lbs.

Pin L_1.—Investigated under floorbeam connection.

Pin L_2.—Fig. 298. *Intermediate Post U_2L_2.*

Thickness of web	$= 0.21$ in.
Thickness of plate	$= 0.375$ in.
Total thickness of bearing	$= 0.585$ in.

Stress in plate $= (28{,}275 \times 0.375)/0.585 = 20{,}000$ lbs.
6 ¾″ rivets transmit the stress in single shear.
Actual shear on one rivet $= 20{,}000/6 = 3{,}330$ lbs.
Allowable shear on one rivet $= 4{,}418$ lbs.

Pin L_3.—Fig. 301. *Intermediate Post U_3L_3.*

The thickness of the bearing is the same as for U_2L_2, while the stress is less.

Pin L_4.—Fig. 303. *Intermediate Post U_4L_4.*

Thickness of web	$= 0.20$ in.
Thickness of plate	$= 0.375$ in.
Total thickness of bearing	$= 0.575$ in.

Stress in plate $= (10{,}950 \times 0.375)/0.575 = 7{,}140$ lbs.
4 ¾″ rivets transmit the stress in single shear.
Actual shear on one rivet $= 7{,}140/4 = 1{,}785$ lbs.
Allowable shear on one rivet $= 4{,}418$ lbs.

BEARING ON RIVETS. **Pin L_0.**—*End-post U_1L_0.* (1) *Bearing of Rivets on Web.*
Shear on one rivet from outside plate $= 1{,}360$ lbs.
Shear on one rivet from inside plate $= 2{,}060$ lbs.
Maximum bearing on one rivet in web $= 1{,}360 + 2{,}060 = 3{,}420$ lbs.
Allowable bearing of a ¾″ rivet on a 0.24″ plate $= \frac{3}{4} \times 0.24'' \times 18{,}000$ $= 3{,}242$ lbs. Not sufficient rivets.

(2) *Bearing on Outside Plate.*—
Actual bearing on one rivet $= 1{,}360$ lbs.
Allowable bearing of a ¾″ rivet on a ⅜″ plate $= 5{,}062$ lbs.

(3) *Bearing on Inside Plate.*—
Actual bearing on one rivet $= 2{,}060$ lbs.
Allowable bearing of a ¾″ rivet on a ¼″ plate $= 3{,}375$ lbs.

Pin U_1.—*End-Post L_0U_1.* (1) *Bearing of Rivets on Web.*
Shear on one rivet from outside plates $= 1,846$ lbs.
Shear on one rivet from inside plate $= 1,400$ lbs.
Total bearing of one rivet in the web $= 1,846 + 1,400 = 3,246$ lbs.
Allowable bearing of a $\frac{3}{4}''$ rivet on a $0.24''$ plate $= 3,242$ lbs.

(2) *Bearing on First Outside Plate.*—
Total bearing of one rivet on a $\frac{1}{4}''$ plate $= 1,120$ lbs.
Allowable bearing of a $\frac{3}{4}''$ rivet on a $\frac{1}{4}''$ plate $= 3,375$ lbs.

(3) *Bearing on Second Outside Plate.*—
Total bearing of one rivet on a $\frac{9}{16}''$ plate $= 1,846$ lbs.
Allowable bearing of a $\frac{3}{4}''$ rivet on a $\frac{9}{16}''$ plate $= 7,594$ lbs.

(4) *Bearing on Inside Plate.*—
Total bearing of one rivet on a $\frac{3}{8}''$ plate $= 1,400$ lbs.
Allowable bearing of one rivet on a $\frac{3}{8}''$ plate $= 5,062$ lbs.

Top Chord U_1U_2. (1) *Bearing of Rivets on Web.*—
Shear of one rivet from outside plate $= 920$ lbs.
Shear of one rivet from inside plate $= 3,077$ lbs.
Total bearing of rivets in web $= 920 + 3,077 = 3,997$ lbs.
Allowable bearing of a $\frac{3}{4}''$ rivet on a $0.24''$ plate $= 3,242$ lbs.

(2) *Bearing of Rivets on Outside Plate.*—
Total bearing of rivet on a $\frac{3}{8}''$ plate $= 920$ lbs.
Allowable bearing of a $\frac{3}{4}''$ rivet on a $\frac{3}{8}''$ plate $= 5,062$ lbs.

(3) *Bearing of Rivets on First Inside Plate.*—
Total bearing of one rivet on a $\frac{1}{4}''$ plate $= 1,230$ lbs.
Allowable bearing of one $\frac{3}{4}''$ rivet on a $\frac{1}{4}''$ plate $= 3,375$ lbs.

(4) *Bearing of Rivets on Second Inside Plate.*—
Total bearing of one rivet on a $\frac{3}{8}''$ plate $= 3,077$ lbs.
Allowable bearing of a $\frac{3}{4}''$ rivet on a $\frac{3}{8}''$ plate $= 5,062$ lbs.

Pin U_2. *Top Chord U_1U_2 and U_2U_3.* (1) *Bearing of Rivets on Web.*—
Total bearing of one rivet on a $0.24''$ plate $= 680$ lbs.
Allowable bearing of a $\frac{3}{4}''$ rivet on a $0.24''$ plate $= 3,242$ lbs.

(2) *Bearing Rivets in Pin Plate.*—
Total bearing of one rivet on a $\frac{1}{4}''$ plate $= 680$ lbs.
Allowable bearing of a $\frac{3}{4}''$ rivet on a $\frac{1}{4}''$ plate $= 3,375$ lbs.

Intermediate Post U_2L_2. (1) *Bearing of Rivets on Web.*—
Total bearing of one rivet on a $0.21''$ plate $= 1,500$ lbs.
Allowable bearing of a $\frac{3}{4}''$ rivet on a 0.21 plate $= 2,923$ lbs.

(2) *Bearing of Rivets on Plate.*—
Total bearing of one rivet on a $\frac{1}{2}''$ plate = 1,500 lbs.
Allowable bearing of a $\frac{3}{4}''$ rivet on a $\frac{1}{2}''$ plate = 6,750 lbs.

Pin U_3. *Top Chord $U_2U_3U_4$.*—See splice on page 508.

Intermediate Post U_3L_3. (1) *Bearing of Rivets on Web.*—
Total bearing of one rivet on a 0.21″ plate = 1,163 lbs.
Allowable bearing of a $\frac{3}{4}''$ rivet on a 0.21″ plate = 2,923 lbs.

(2) *Bearing of Rivets on Pin Plate.*—
Total bearing of one rivet on a $\frac{3}{8}''$ plate = 1,163 lbs.
Allowable bearing of a $\frac{3}{4}''$ rivet on a $\frac{3}{8}''$ plate = 5,062 lbs.

Pin U_4. *Top Chord $U_3U_4U_3'$.* (1) *Bearing of Rivets on Web.*—
Total bearing of one rivet on a 0.24″ plate = 1,240 lbs.
Allowable bearing of a $\frac{3}{4}''$ rivet on a 0.24″ plate = 3,242 lbs.

(2) *Bearing of Rivets on Pin Plate.*—
Total bearing of one rivet on a $\frac{1}{4}''$ plate = 1,240 lbs.
Allowable bearing of a $\frac{3}{4}''$ rivet on a $\frac{1}{4}''$ plate = 3,375 lbs.

Intermediate Post U_4L_4. (1) *Bearing of Rivets in the Web.*—
Total bearing of one rivet on a 0.20″ plate = 575 lbs.
Allowable bearing of a $\frac{3}{4}''$ rivet on a 0.20″ plate = 2,700 lbs.

Pin M_1. *Hip Vertical U_1M_1.* (1) *Bearing of Rivets in the Web.*—
Total bearing of one rivet on a 0.20″ plate = 1,521 lbs.
Allowable bearing of a $\frac{3}{4}''$ rivet on a 0.20″ plate = 2,700 lbs.

(2) *Bearing of Rivets on Pin Plate.*—
Total bearing of one rivet on a $\frac{1}{4}''$ plate = 1,521 lbs.
Allowable bearing of a $\frac{3}{4}''$ rivet on a $\frac{1}{4}''$ plate = 3,375 lbs.

Pin L_2. *Intermediate Post U_2L_2.* (1) *Bearing of Rivets on the Web.*—
Total bearing of one rivet on a 0.21″ plate = 3,330 lbs.
Allowable bearing of a $\frac{3}{4}''$ rivet on a 0.21″ plate = 2,923 lbs.

(2) *Bearings of Rivets on Pin Plate.*—
Total bearing of one rivet on a $\frac{3}{8}''$ plate = 3,330 lbs.
Allowable bearing of a $\frac{3}{4}''$ rivet on a $\frac{3}{8}''$ plate = 5,062 lbs.

Pin L_3. *Intermediate Post U_3L_3.*—The dimensions of the web and the pin plate are the same as for U_2L_2, and the stress is less.

Pin L_4. *Intermediate Post U_4L_4.* (1) *Bearing of Rivets on the Web.*—
Total bearing of one rivet on a 0.20″ plate = 1,785 lbs.
Allowable bearing of a $\frac{3}{4}''$ rivet on a 0.20″ plate = 2,700 lbs.

(2) *Bearing of Rivets on Pin Plate.*—
Total bearing of one rivet on a ⅜″ plate = 1,785 lbs.
Allowable bearing of a ¾″ rivet on a ⅜″ plate = 5,062 lbs.

SPACING OF RIVETS.—(a) The distance between lines of rivets perpendicular to the length of the member shall not be greater than 40 times the thickness of the cover plate (§ 67).

Member.	Actual.	Allowable.
End-post L_0U_1	9½″	10″
Top Chord $U_1U_2 - U_1'$	9½	10

(b) The pitch of the rivets in the line of the stress shall not exceed 6″ nor 16 times the thickness of the thinnest outside plate, nor be less than 3 diameters of the rivet (§ 62).

Member.	Maximum.		Minimum.	
	Actual.	Allowable.	Actual.	Allowable.
End-post L_0U_1	5″	4″ to 2¼″	2½″	2¼″
Top Chord $U_1U_2 - U_1'$	5	4 to 2¼	3	2¼

(c) The pitch of the rivets at the ends of compression members shall not exceed 4 diameters of rivets for a length equal to one and one-half times the width of the member (§ 66).

Member.	Actual.		Required.	
	Pitch.	Length.	Pitch.	Length.
End-post L_0U_1	2½″	1′ 6½″	2¼″ to 3″	1′ 6″
Top Chord $U_1U_2 - U_1'$	3	1 6	2¼ to 3	1 6

(d) The distance between the edge of the piece and the center of the rivet must not be less than 1¼″ for ¾″ rivets and 1⅛″ for ⅝″ rivets (§ 63).

Member.	Actual.	Required.
End-post L_0U_1	1¼″	1¼″
Top Chord $U_1U_2 - U_1'$	1¼	1¼
Int. Post U_2L_2	1⅛	1⅛
Int. Post U_3L_3	1⅛	1⅛
Int. Post U_4L_4	1	1⅛
Hip Vertical M_1L_1	¾	1⅛
Portal	1⅛	1⅛

BATTEN PLATES.—Batten plates must be placed as near the ends of compression members as possible and shall have a length not less than the greatest width of the member nor 1½ times the least width (§ 69).

Member.	Actual.		Required.	
	Length.	Pitch of Rivets.	Length.	Pitch of Rivets.
End-post L_0U_1	12″ & 10″	2½″ to 3″	15″	2¼″ to 3″
Top Chord $U_1U_2 - U_1'$	12	3	15	2¼ to 3
Inter. Post U_2L_2 & U_3L_3	10	2½	10½	2¼ to 3
Inter. Post U_4L_4	9½	2½	10	2¼ to 3

LACING BARS.—(a) Lacing bars must not be less in width than 1½″ for members 6″ wide, 1¾″ for members 9″ wide, 2″ for members 12″ wide, etc. (§ 70).

(b) Single lattice bars shall have a thickness not less than ¹⁄₄₀ of the distance between rivets, and double lacing shall have a thickness not less than ¹⁄₆₀ of the distance between rivets (§ 70).

(c) Lattice bars shall be inclined at an angle of not less than 60° with the axis of the member for single lacing or 45° for double lacing (§ 70).

(d) The pitch of the lacing must not exceed the width of the member plus 9″ (§ 71).

Member.	Actual.				Required.			
	Width. *a*	Thickness. *b*	Angle. *c*	Pitch. *d*	Width. *a*	Thickness. *b*	Angle. *c*	Pitch. *d*
End-post L_0U_1	2″	⅜″	57°	12″	2″	0.28″	60°	19″
Top Chord U_1U	2	⅜	57	12	2	0.28	60	19
Int. Post U_2L_2	1¾	¼	59	10	1¾	0.24	60	16
Int. Post U_4L_4	1½	¼	58	10	1½	0.24	60	15

JOISTS.—The joist consists of two lines of [s 7″ @ 9¾ lbs. and seven lines of Is 7″ @ 15 lbs., spaced 1′ 11¼″ (use 2′ 0″ in calculations) and have a span of 20′ 0″.

(a) *Uniform Load.*—The dead load per foot of bridge exclusive of the weight of joists $= 16 \times 2\frac{1}{2} \times 4\frac{1}{2} + (2 \times 4 \times 6 \times 4\frac{1}{2})\frac{1}{12} + (3 \times 4 \times 6 \times 4\frac{1}{2})\frac{1}{12} = 225$ lbs. Dead load per foot of joist exclusive of the weight of joists = 30 lbs. Live load per foot of joist = 200 lbs. Total load per foot of joist exclusive of the weight of joist = 230 lbs. The weight of the joist is 15 lbs. per lineal foot of joist, which is less than

10 per cent of the total load, and may be neglected (§ 48). The maximum bending moment $= \frac{1}{8} \times 230 \times 20^2 \times 12 = 138{,}000$ in.-lbs.

Actual unit bending stress $= S = Mc/I = (138{,}000 \times 3\frac{1}{2})/36.2 = 13{,}340$ lbs. Allowable unit stress $= 13{,}000$ lbs.

Efficiency $= 13{,}000/13{,}340 = 0.96$.

(b) *Concentrated Loads,* Fig. 304.—The concentrated load is a traction engine weighing 15,800 lbs., two-thirds of the load being car-

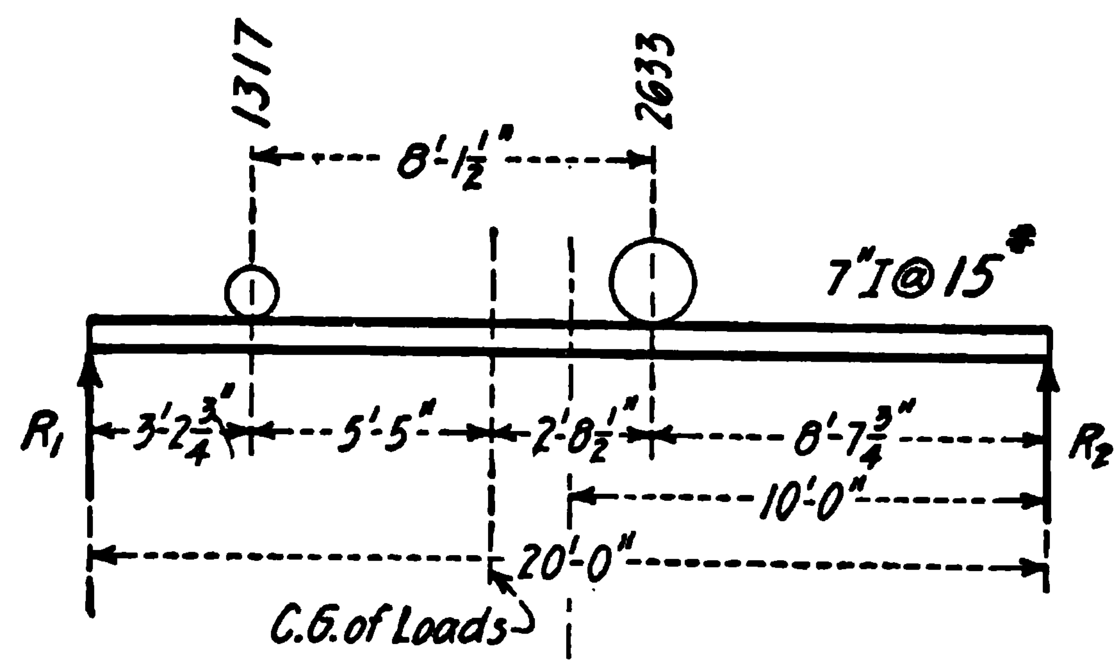


FIG. 304.

ried on the rear axle. The distance c to c of axles $= 8'\ 1\frac{1}{2}''$, distance c to c of front wheels $= 4'\ 7\frac{1}{2}''$, c to c of rear wheels $= 5'\ 10''$; width of front wheels $= 6''$; width of rear wheels $= 16''$. It will be assumed that the front and rear wheels may be so nearly in line that a joist will carry the same load as when this condition occurs, and that the floor is stiff enough to so distribute the load that the load on one side will be carried by two joists.

The loads coming on one joist will be 2,633 and 1,317 lbs. The condition for maximum bending moment in the joist is *that the heavier wheel under which the moment will occur will be as far from one end of the beam as the center of gravity of the two loads is from the other end of the beam.* The heavier load will be $8'\ 7\frac{3}{4}''$ from one end, as shown in Fig. 304. From Fig. 304 $R_2 = (1{,}317 \times 3.23 + 2{,}633 \times 11.35)/20 = 1{,}705$ lbs.

The maximum bending moment is $M = 1{,}705 \times 8.65 \times 12 = 176{,}600$ in.-lbs.

The bending moment at the same point due to dead load, dead load $= 30$ lbs. per foot of joist, will be $M = 300 \times 11.35 - \frac{1}{2} \times 30 \times \overline{11.35}^2 = 15{,}940$ in.-lbs.

Total bending moment due to dead and live loads = 176,600 + 15,940
= 192,540 in.-lbs.
Actual unit stress = $S = Mc/I$ = (192,540 × 3½")/36.2 = 18,500 lbs.
Allowable unit stress = 13,000 lbs.
Efficiency = 13,000/18,500 = 0.70.

FLOORBEAMS.—The floorbeams are Is 15"@ 50 lbs., 16' 5" long. The clear span will be assumed to be 16' 5".

(a) *Uniform Load.*—

Dead load of floor per lineal foot of bridge	= 225 lbs.
Weight of joist per lineal foot of bridge, 2 lines 7" [s @ 9¾ lbs.	= 19 lbs.
Weight of joist per lineal foot of bridge, 7 lines 7" Is @ 15 lbs.	= 105 lbs.
Total dead load per lineal foot of bridge	= 345 lbs.

Dead load carried by one floorbeam = 345 × 20 = 6,900 lbs.
Weight of floorbeam = 16' 5" × 50 lbs. = 825 lbs.

The weight of the floorbeam is less than 10 per cent of the total live and dead load, and may be neglected (§ 48).

Live load carried by one floorbeam = 17 × 20 × 100 = 34,000 lbs.
Total dead and live load carried by one floorbeam = 40,900 lbs.
Maximum bending moment = $\frac{1}{8} \times W \times l = \frac{1}{8}$ × 40,900 × 16' 5" × 12
= 966,876 in.-lbs.
Actual unit stress = $S = M \cdot c/I$ = (966,876 × 7½)/483.4 = 15,000 lbs.
Allowable unit stress = 13,000 lbs.
Efficiency = 13,000/15,000 = 0.87.

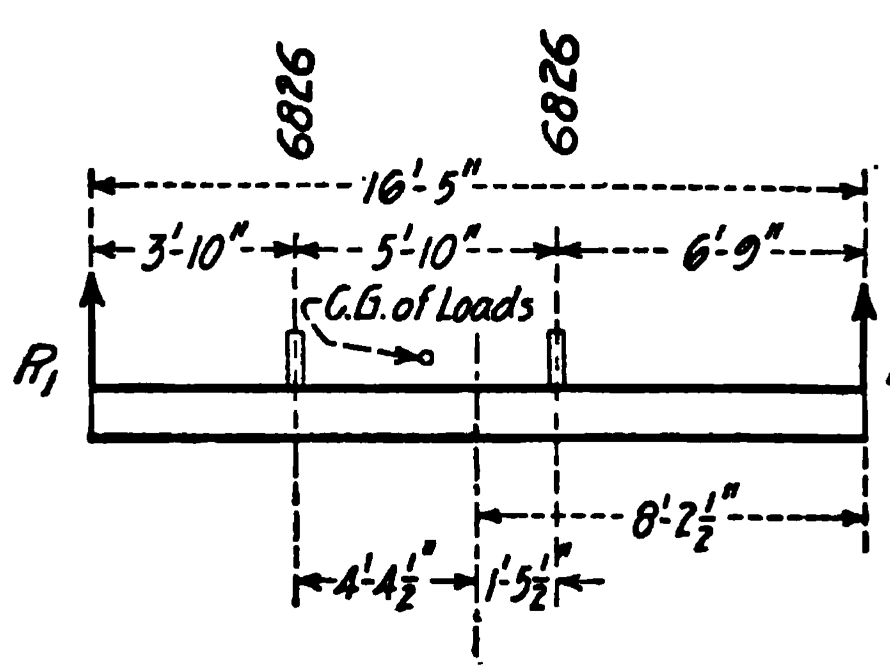


FIG. 305.

(b) *Concentrated Loads.*—The concentrated load is the same traction engine as was used in investigating the joists.

The large wheels, Fig. 305, will be directly over the floorbeam *with one of the wheels as far from one end of the floorbeam as the center of gravity of the two wheels is from the other end of the floorbeam.*

The maximum bending moment will occur under the wheel that is nearest the center. The reaction at each wheel will be $= 5,267 + (2,633 \times 11.87)/20 = 6,826$ lbs.

The reaction at the right end is $R_2 = [6,826 \times (3.83 + 9.67)]/16.416 = 5,610$ lbs.

The maximum bending moment $= M = 5,610 \times 81'' = 454,500$ in.-lbs.

Dead load bending moment at the same point $= 3,450 \times 81'' - (3,450 \times 81 \times 6\frac{3}{4})/(16,416 \times 12) = 164,450$ in.-lbs.

Total bending moment $= 454,500 + 164,450 = 618,950$ in.-lbs.

Actual unit stress $= S = M \cdot c/I = (618,950 \times 7\frac{1}{2})/483.4 = 9,650$ lbs.

Allowable fiber stress $= 13,000$ lbs.

Efficiency $= 13,000/9,650 = 1.45$.

Floorbeam Connection.—Floorbeams L_1, L_2, L_3 and L_4.

Maximum end shear including weight of floorbeam $= \frac{1}{2} \times 41,725 = 20,863$ lbs.

Connection angles $=$ 2 angles $2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{4}'' \times 1'\ 10''$ long.

(1) **Rivets in Floorbeams.**

(A) *Bearing.*

(a) *In Web of Floorbeam.*—Thickness of web $= 0.56''$; 10. $\frac{3}{4}''$ shop rivets.

Bearing on one rivet $= 20,863/10 = 2,086$ lbs.

Allowable bearing of one rivet on a $0.56''$ plate $= 6,075$ lbs.

(b) *On Plates.*—Ten $\frac{3}{4}''$ rivets in each plate.

Bearing on one rivet $= 20,863/20 = 1,043$ lbs.

Allowable bearing of one rivet on a $\frac{1}{4}''$ plate $= 3,375$ lbs.

(B) *Shear.*—Shear on one rivet $= 1,043$ lbs.

Allowable shear on one $\frac{3}{4}''$ rivet $= 3,554$ lbs.

(2) **Rivets in Connection Angles.**

(A) *Bearing.*—Ten $\frac{3}{4}''$ field rivets.

(a) *Bearing on the Two $\frac{1}{4}''$ Connection Plates.*—Bearing on one rivet $= 20,863/10 = 2,086$ lbs. Allowable bearing of a $\frac{3}{4}''$ field rivet on two $\frac{1}{4}''$ plates $= 3,600$ lbs.

(b) *Bearing on Connection Angles.*—Actual bearing on one rivet $= 20,863/20 = 1,043$ lbs.

Allowable bearing of one $\frac{3}{4}''$ field rivet on a $\frac{1}{4}''$ plate $= 1,800$ lbs.

(B) *Shear.*—Shear on one ¾″ field rivet = 20,863/20 = 1,043 lbs. Allowable shear on one ¾″ field rivet = 2,356 lbs.

(3) **Rivets in Posts.**

(A) *Bearing.*—Eight ¾″ shop rivets on each side.

(a) *Bearing on Web of Channel.*—Thickness of web = 0.20″.
Bearing of one rivet on the web = 20,863/16 = 1,304 lbs.
Allowable bearing of one ¾″ rivet on a 0.20″ plate = 2,160 lbs.

(b) *Bearing on Angles.*—Bearing of one rivet on the angle = 20,863/16 = 1,304 lbs.
Allowable bearing of one ¾″ rivet on a ¼″ plate = 2,700 lbs.

(B) *Shear.*—The shear on one rivet = 20,863/16 = 1,304 lbs.
Allowable shear of one ¾″ rivet = 3,534 lbs.

LOWER LATERAL SYSTEM. Panel L_0L_1.—One rod 1⅜″ ϕ × 25′ 8″ long. Area of rod = 1.485 sq. in.
Maximum stress in member = 36,630 lbs.
Actual unit stress = 36,630/1.485 = 24,700 lbs.
Allowable unit stress = 18,000 lbs. (§36a).
The rod should have an area of 2.04 sq. in., which would require a 1⅝″ ϕ rod.

Panel L_1L_2.—One rod 1¼″ ϕ × 25′ 9″ long. Area of rod = 1.28 sq. in.
Maximum stress in member = 28,305 lbs.
Actual unit stress = 28,305/1.28 = 23,000 lbs.
Allowable unit stress = 18,000 lbs.
The rod should have an area of 1.57 sq. in., which would require a 1$\frac{7}{16}$″ ϕ rod.

Panel L_2L_3.—One rod 1⅛″ ϕ × 25′ 9″ long. Area of rod = 0.99 sq. in.
Maximum stress in member = 18,315 lbs.
Actual unit stress = 18,315/0.99 = 18,500 lbs.
Allowable unit stress = 18,000 lbs.

Panel L_3L_4.—One rod 1″ ϕ × 25′ 2″ long. Area of rod = 0.785 sq. in.
Maximum stress in member = 8,880 lbs.
Actual unit stress = 8,880/0.785 = 11,300 lbs.
Allowable stress = 18,000 lbs.

Lateral Pin at L_0.—The pin is 1$\frac{5}{16}$″ diameter.
Stress in pin = 36,630 lbs.

Bearing on Pin.—Assume rod flattened to 1" wide.
Area of bearing on pin $= 1'' \times 1.3125'' = 1.3125$ sq. in.
Actual unit bearing stress $= 36{,}630/1.3125 = 27{,}900$ lbs.
Allowable unit bearing $= 18{,}000 \times 1.40 = 25{,}200$ lbs.

Shear.—Area of cross-section of the pin $= 2'' \times 1.35'' = 2.70$ sq. in.
Actual unit shear $= 36{,}630/2.70 = 13{,}560$ lbs.
Allowable unit shear $= 10{,}000 \times 1.40 = 14{,}000$ lbs.

Lateral Connection.—The lateral connection on the floorbeam, at L_1, receives a maximum stress. The maximum stress $= 24{,}750$ lbs.
6 $\frac{3}{4}''$ rivets take the stress in single shear.
Actual shear on one rivet $= 24{,}750/6 = 4{,}125$ lbs.
Allowable shear on one $\frac{3}{4}''$ rivet $= 6{,}185$ lbs.

UPPER LATERAL SYSTEM. (1) **Diagonal Rods. Panel U_1U_2.**—One rod $1\frac{1}{4}'' \phi \times 26'\ 4\frac{1}{2}''$ long. Area of rod $= 1.23$ sq. in.
Maximum stress $= 5{,}550$ lbs.
Actual unit stress $= 5{,}550/1.23 = 4{,}570$ lbs.
Allowable unit stress $= 18{,}000$ lbs.

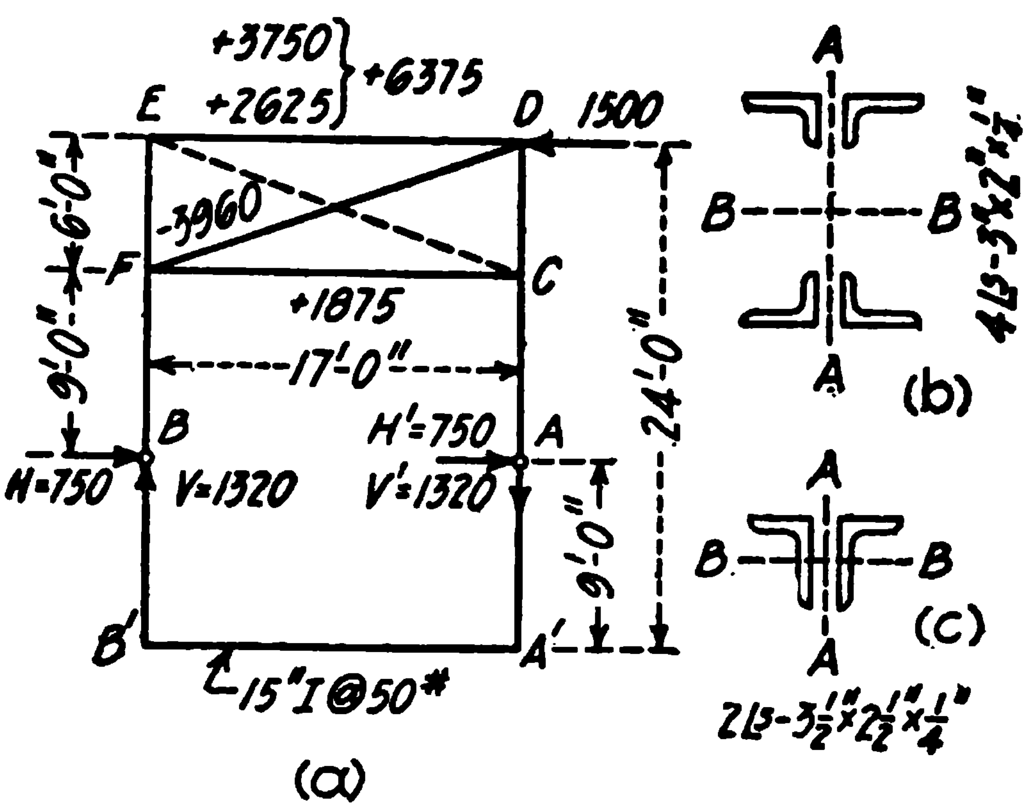


FIG. 306. SWAY BRACING.

Panel U_2U_3.—One rod $1\frac{1}{8}'' \phi \times 26'\ 4''$ long. Area of rod $= 0.99$ sq. in.
Maximum stress $= 3{,}330$ lbs.
Actual unit stress $= 3{,}330/0.99 = 3{,}400$ lbs.
Allowable unit stress $= 18{,}000$ lbs.

Panel U_3U_4.—One rod $1'' \phi \times 26'\ 2\frac{1}{2}''$ long. Area of rod $= 0.785$ sq. in.
Maximum stress $= 1{,}110$ lbs.

Actual unit stress = 1,110/0.785 = 1,430 lbs.
Allowable unit stress = 18,000 lbs.

Lateral Connections. *Connection at U_1.*—
Stress = 5,550 lbs. The stress is transmitted by 6 $\frac{3}{4}''$ rivets in single shear.
Actual shear on one rivet = 5,550/6 = 926 lbs.
Allowable shear on one $\frac{3}{4}''$ rivet = 6,185 lbs.

(2) **Top Lateral Struts. Strut U_2U_3** carries a maximum stress and will be analyzed. The top lateral strut is composed of 4 angles $3'' \times 2\frac{1}{2}'' \times \frac{1}{4}'' \times 16' \cdot 0''$.

Moment of inertia about axis A–A.

$$I = I' + Ad^2 = 4(1.09 + 1.19 \times 1.24^2)$$
$$= 11.68''^4$$
$$r = \sqrt{11.68/4.76} = 1.57''; \; l/r = 192/1.57 = 123.$$

Actual unit stress = 6,375/4.76 = 1,340 lbs.
Allowable unit stress = 13,000 − 60·l/r = 5,660 lbs.

Rivets at the End.—The stress is carried by 12 $\frac{5}{8}''$ field rivets. These will carry a shear of 12 × 2,863 = 34,356 lbs. (Table LXXVI), which is ample.

(3) **Sway Strut.**—Sway strut A–B at U_2 will be considered only. The member is composed of 2 angles $3\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{4}'' \times 16' \; 5\frac{1}{2}''$.
Radius of gyration about axis A–A = 1.04″.
Radius of gyration about axis B–B = 1.12″.
l/r = 190 (too large, should not exceed 150, § 37a).
Actual unit stress = 1,875/3.12 = 600 lbs.
Allowable unit stress = 13,000 − 60·l/r = 1,600 lbs.
The member is poorly designed.

(4) **Sway Rods.**—One rod $\frac{7}{8}''$ ϕ. Area of one rod = 0.60 sq. in.
Maximum stress = 3,960 lbs.
Actual unit stress = 3,960/0.60 = 6,600 lbs.
Allowable unit stress = 18,000 lbs.

PORTAL. *Member F–E–D.* Member is composed of 2 angles $3\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{5}{16}''$.
Radius of gyration about axis A–A = 1.68″.
Radius of gyration about axis B–B = 0.73″.
Ratio of l/r about axis B–B = 96/0.73 = 131.
Ratio of l/r about axis A–A = 192/1.68 = 115.
Ratio of l/r should not exceed 150 (§ 37a).

Allowable unit stress $= P = 13{,}000 - 60 \cdot l/r = 5{,}140$ lbs. (§ 37a).
Maximum compressive stress $= 15{,}880 + 0.8(9{,}150) = 23{,}200$ lbs. (§ 42a).
Actual unit compressive stress $= 23{,}200/3.56 = 6{,}500$ lbs.
Maximum tensile stress $= 9{,}130 + 0.8(9{,}130) = 16{,}434$ lbs. (§ 42a).

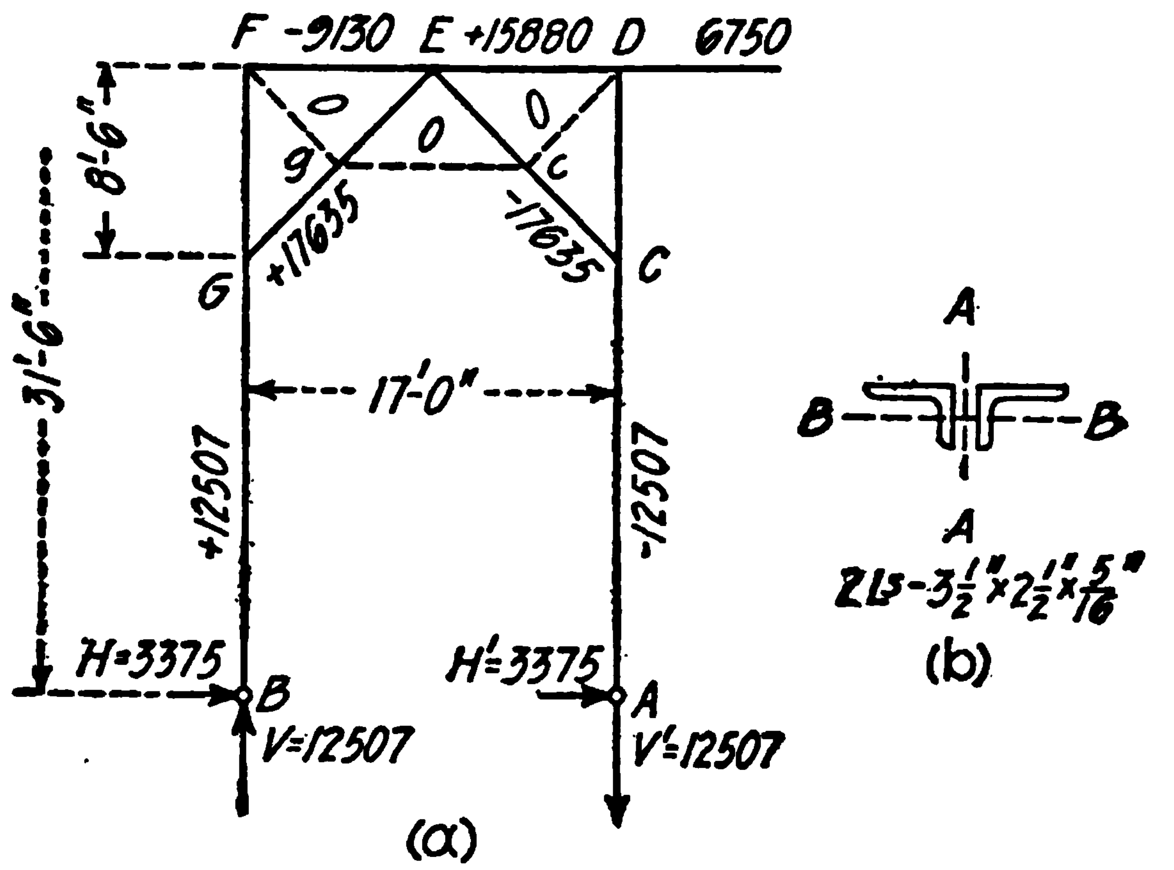


FIG. 307. PORTAL.

Net area $= 3.56 - 0.44 = 3.12$ sq. in.
Actual unit tension $= 16{,}434/3.12 = 5{,}270$ lbs.
Allowable unit stress $= 18{,}000$ lbs. (§ 36a).

Members G–E and E–C.—Same section as *F–E–D.*
Maximum compression $= 17{,}635 + 0.8(17{,}635) = 31{,}743$ lbs. (§ 42a).
Ratio of l/r about axis B–$B = 72/0.73 = 100$.
Ratio of l/r about axis A–$A = 144/1.68 = 86$.
Allowable stresses $= P = 13{,}000 - 60 \cdot l/r = 7{,}000$ lbs. (§ 37a).
Actual unit stress $= 31{,}743/3.56 = 8{,}900$ lbs.
Maximum tensile stress $= 31{,}743$ lbs.
Net section $= 3.56 - 0.44 = 3.12$ sq. in.
Actual unit tension $= 31{,}743/3.12 = 10{,}170$ lbs.
Allowable unit stress $= 18{,}000$ lbs. (§ 36a).
Rivets at the foot of knee brace. Five $\frac{3}{4}''$ field rivets take a stress of 31,743 lbs. in single shear (§ 42a).
Actual shear on one rivet $= 31{,}743/5 = 6{,}345$ lbs.
Allowable shear on one $\frac{3}{4}''$ field rivet $= 4{,}123$ lbs. (Table LXXVII).

Rivets at the Middle of the Top Strut.—

Five ¾″ shop rivets take a stress of 31,743 lbs. in single shear (§ 42a).
Actual shear on one rivet = 31,743/5 = 6,350 lbs.
Allowable shear on one ¾″ shop rivet = 6,185 lbs. (Table LXXVII).
Bearing on the ¼″ plate. The bearing on the ¼″ plate = 7,935 lbs.
The allowable bearing of a ¾″ rivet on a ¼″ plate = 4,725 lbs.

Rivets at End of F–D.—

Twelve ¾″ field rivets take a stress of 23,200 lbs. in single shear (§ 42a).
Actual shear on one rivet = 23,200/12 = 1,940 lbs.
Allowable shear on one ¾″ field rivet = 4,123 lbs.
Bearing on the ¼″ plate = 1,940 lbs. The allowable bearing of one ¾″ field rivet on a ¼″ plate = 3,150 lbs.

PEDESTALS. (1) *Bearing of Pin on the Web* (see pin L_0).

(2) (§ 86 and 87): Vertical webs on the bolster plates and connections are not as specified.
Base plates and webs are ½″ thick or more as specified.
Bed plates are bolted to masonry as specified by 1¼″ bolts.

ROLLERS.—Each nest is composed of 6 2″ rollers 1′ 3¾″ long.
Minimum diameter = 3.6″ (§ 84).
Load on the rollers = 76,650 lbs.
Total length of rollers = 6 × 15¾″ = 94½″.
The load per lineal inch = 76,650/94½ = 811 lbs.
Allowable pressure per lineal inch of rollers = 2 × 300 = 600 lbs.

The rollers are insufficient.

Top Chord Splice.—§ 73 requires that there be two rows of closely spaced rivets on each side of the splice, which is satisfied.

The student should compare the bridge with the specifications in Appendix I, in detail; and should prepare a tabulated report on the efficiencies of the different parts.

APPENDIX I.

General Specifications for Steel Highway Bridges.

By Milo S. Ketchum, M. Am. Soc. C. E.

Introduction.—In preparing these specifications the author has made use of various standard specifications: Classes of Highway Bridges A, B and C as used by Theodore Cooper and the American Bridge Company have been adopted; while their Class D has been divided into Class D_1 and D_2. The classes of electric railway bridges used by C. C. Schneider in his "Specifications for Electric Railway Bridges" have been adopted, and have been named E_1, E_2 and E_3. The allowable stresses and the specifications for materials and workmanship as given in the Standard Bridge Specifications of the American Railway Engineering and Maintenance of Way Association have been adopted with slight necessary modifications. The impact coefficient for highway bridges has been taken as one-half the value prescribed for railway bridges.

The author has used the stresses and the method of dimensioning given in Cooper's "General Specifications for Steel Highway and Electric Railway Bridges and Viaducts" in the "Calculation of the Efficiencies of the Members of a Steel Highway Bridge," Part III, and has therefore given Cooper's stresses as alternate allowable stresses. Either set of allowable stresses may be used in designing highway bridges.

Attention is called to the fact that a minimum thickness of metal of ¼ in., and a maximum ratio of l/r in compression members of 125 and 150 in main and lateral members, respectively, are allowed in Class D_1 and Class D_2 bridges.

GENERAL SPECIFICATIONS FOR STEEL HIGHWAY BRIDGES.

PART I. DESIGN.

General Description.

1. **Classes.**—Bridges under these specifications are divided into eight classes, as follows:

Class A.—For city traffic.
Class B.—For suburban or interurban traffic with heavy electric cars.
Class C.—For country roads with ordinary traffic and light electric cars.
Class D_1.—For country roads with heavy traffic.
Class D_2.—For country roads with light traffic.
Class E_1.—For heavy electric street railways only.
Class E_2.—For medium electric street railways only.
Class E_3.—For light electric street railways only.

2. **Material.**—All parts of the structure shall be of rolled steel, except the flooring, floor joists and wheel guards, when wooden floors are used. Cast iron

or cast steel may be used in the machinery of movable bridges for wheel guards, and in special cases for bed plates.

3. **Types of Truss.**—The following types of bridges are recommended:

Spans up to 30 feet—Rolled beams.

Spans from 30 to 80 feet—Riveted plate girders, or riveted low trusses for classes A, B, E_1, E_2 and E_3; and riveted low trusses for classes C, D_1 and D_2.

Spans 80 to 160 feet—Riveted or pin-connected high trusses.

Spans 160 to 200 feet—Pin-connected trusses of the Pratt type with inclined chords.

Spans over 200 feet—Pin-connected trusses of the Petit type.

4. **Length of Span.**—In calculating the stresses the length of span shall be taken as the distance between centers of end pins for pin-connected trusses, centers of end bearing plates for riveted trusses and for girders, and center to center of trusses for floorbeams.

5. **Form of Trusses.**—The form of truss shall preferably be as given in paragraph 3. In through trusses the end vertical suspenders and the two panels of the lower chord at each end shall be made rigid members if the wind load produces a reversal of stress in the lower chord. In through bridges the floorbeams shall be riveted above or below the lower chord pins.

6. **Lateral Bracing.**—All lateral and sway bracing shall preferably, and all portal bracing must be, made of shapes capable of resisting compression as well as tension, and shall have riveted connections. Low trusses and through plate girders shall be stayed by knee braces or gusset plates at each floorbeam.

7. **Spacing of Trusses.**—For bridges carrying electric cars the clear width from the center of the track shall not be less than 7 feet at a height exceeding one foot above the track where the tracks are straight, and an equivalent distance when the tracks are curved. The distance between centers of trusses shall in no case be less than one-twentieth of the span between the centers of end-pins or shoes, and shall preferably not be less than one-twelfth of the span.

8. **Head Room.**—For classes A, B, C, D_1, E_1, E_2 and E_3 the clear head room for a width of six (6) feet on each track, or the center of the bridge shall not be less than 15 feet, and for class D_2 not less than 12½ feet.

9. **Footwalks.**—Where footwalks are required, they shall generally be placed outside of the trusses and be supported on longitudinal beams resting on overhanging steel brackets.

10. **Handrailing.**—A strong and suitable handrailing shall be placed at each side of the bridge and be rigidly attached to the superstructure.

11. **Trestle Towers.**—Trestle bents shall preferably be composed of two supporting columns, two bents forming a tower; each tower thus formed shall be thoroughly braced in both directions and have struts between the feet of the columns. The feet of the columns must be secured to an anchorage capable of resisting one and one-half times the specified wind forces (§ 89).

Each tower shall have a sufficient base, longitudinally to be stable when standing alone, without other support than its anchorage. Tower spans for high trestles shall not be less than 30 feet.

12. **Proposals.**—Contractors in submitting proposals shall furnish complete stress sheets, general plans of the proposed structures, and such detail drawings as will clearly show the dimensions of all the parts, modes of construction and sectional areas.

13. **Drawings.**—Upon the acceptance and the execution of the contract, all working drawings required by the engineer shall be furnished free of cost (§ 168).

14. **Approval of Plans.**—No work shall be commenced or materials ordered until the working drawings have been approved by the engineer in writing.

FLOOR SYSTEM.

15. **Floorbeams.**—All floorbeams shall be rolled or riveted steel girders, rigidly connected to the trusses at the panel points, or may be placed on the top of deck bridges at panel points. Floorbeams shall preferably be square to the trusses or girders.

16. **Joists and Stringers.**—All joists and stringers of bridges of classes A, B, E_1, E_2 and E_4 shall be of steel. Bridges of classes C, D_1 and D_2 may be either of wood or steel as specified. Steel joists shall be securely fastened to the cross floorbeams, and steel stringers shall preferably be riveted to the webs of floorbeams by means of connection angles at least $\frac{7}{16}$ in. thick.

17. **End Spacers for Stringers.**—Where end floorbeams cannot be used, stringers resting on masonry shall have cross-frames near their ends. These frames shall be riveted to girder or truss shoe where practicable.

18. **Wooden Joists.**—Wooden floor joists shall be spaced not more than $2\frac{1}{2}$ feet centers, and shall lap by each other so as to have a full bearing on the floorbeams, and shall be separated $\frac{1}{2}$ inch for free circulation of air. Their width shall not be less than 3 inches, or one-fourth the depth in width. When spaced not more than 2 feet centers, one joist shall be considered as carrying one-half of the concentrated live load. Oak, longleaf yellow pine and Oregon fir are to be designed for a safe bending of 1,200 lbs. per sq. in., bearing across the fiber of 350 lbs. per sq. in., and shearing along the fiber of 100 lbs. per sq. in.

19. **Steel Joists.**—Steel beams when used as joists shall have a depth of not less than one-thirtieth of the span, and one-twentieth of the span when used as track stringers. Steel joists shall be spaced not to exceed 3 feet centers. When spaced not to exceed 2 feet centers, one joist shall be considered as carrying one-half the concentrated load; when spaced more than 2 feet and not more than 3 feet one joist shall be considered as carrying two-thirds of the concentrated load.

20. **Floor Plank.**—For single thickness the roadway planks shall not be less than $2\frac{1}{2}$ inches thick for oak or 3 inches for pine, nor less than one-twelfth of the distance between centers of joists, and shall be laid transversely with $\frac{1}{2}$ inch openings and securely spiked to each joist. All plank shall be laid with heart side down. When an additional wearing surface is required it shall be $1\frac{1}{2}$ inches thick, and the lower planks of a minimum thickness of $2\frac{1}{2}$ inches shall be laid diagonally with $\frac{1}{2}$ inch openings.

21. Footwalk plank shall be not less than 2 inches thick nor more than 6 inches wide, spaced with $\frac{1}{2}$ inch openings.

All plank shall be laid with heart side down, shall have full and even bearing on and be firmly attached to the joists.

22. **Wheel Guards.**—Wheel guards of a cross-section of not less than 6 inches by 4 inches shall be provided on each side of the roadway. They shall be spliced with half-and-half joints with 6 inches lap, and shall be bolted to the stringers or joist with $\frac{5}{8}$ inch bolts, spaced not to exceed 5 feet apart.

23. **Solid Floor.**—For bridges of classes A and B a solid floor, consisting of wooden blocks, brick, stone, asphalt, etc., on a concrete bed is recommended. For this case the floor shall consist of buckle plates or corrugated sections or other satisfactory reinforcement, and a waterproof concrete (bitumen or cement) bed not less than 3 inches thick for the roadway and 2 inches thick for the footwalk, over the highest point to be covered, not counting rivet or bolt heads. The floor shall be laid with a slope of at least one inch in 10 feet.

24. Buckle plates shall not be less than $\frac{5}{16}$ inch thick for the roadway and $\frac{1}{4}$ inch thick for the footwalk. The crown of the plates shall not be less than 2 inches.

25. For solid floor the curb holding the paving and acting as a wheel guard on each side of the roadway shall be of stone or steel projecting about 6 inches above the finished paving at the gutter. The curb shall be so arranged that it can be removed and replaced when worn or injured. There shall also be a metal edging strip on each side of the footwalk to protect and hold the paving in place.

26. **Drainage.**—Provision shall be made for drainage clear of all parts of the metal work.

27. **Floor of Classes E_1, E_2, and E_3.**—The floors of classes E_1, E_2, and E_3 shall consist of cross-ties not less than 6 inches by 6 inches for stringers spaced $6\frac{1}{2}$ feet, and larger for greater spacings, they shall be spaced with openings not exceeding 6 inches, shall be notched down $\frac{1}{2}$ inch, and secured to the supporting stringers by $\frac{3}{4}$ inch bolts spaced not over 6 feet apart. The ties shall extend the full width of the bridge on deck bridges, and every other tie shall extend the full width in through bridges to carry the footwalk. Ties shall be designed for the same allowable unit stresses as wooden joists.

There shall be guard timbers not less than 6 inches by 6 inches, or 5 inches by 7 inches, on each side of each track, with their inner faces not less than 9 inches from the center of the rail. They shall be notched 1 inch over every tie, and shall be spliced over a tie with a half-and-half joint with 6 inches lap. Each guard timber shall be fastened to every third tie and at each splice with a $\frac{3}{4}$ inch bolt. All heads or nuts on the upper faces of ties or guards shall be countersunk below the surface of the wood.

PART II. LOADS.

28. **Dead Load.**—The dead load will consist of (1) the weight of the metal, and (2) the weight of the timber in the floor, or of the material other

than steel. In determining the dead load the weight of oak or other hard woods shall be taken at 4½ lbs. per foot board measure, and the weight of pine or other soft woods at 3½ lbs. per foot; the weight of concrete and asphalt at 130 pounds, of paving brick at 150 pounds, and of granite at 160 pounds per cubic foot.

The rails, fastenings, splices and guard timbers of street railway tracks shall be assumed to weigh not less than 100 pounds per lineal foot of track.

29. **Live Load.**—The bridges of different classes shall be designed to carry, in addition to their own weight and that of the floor, a moving load, either uniform or concentrated, or both, as specified below, placed so as to give the greatest stress in each member.

Class A. For City Traffic.—For the floor and its supports, on any part of the roadway or on each of the street car tracks, a concentrated load of 24 tons on two axles 10 feet centers and 5 feet gage (assumed to occupy 12 feet in width for a single line or 22 feet for a double line), and upon the remaining portion of the floor, including walks, a load of 100 pounds per square foot.

Loads for the trusses as per Table I.

Class B. For Suburban or Interurban Traffic.—For the floor and its supports, on any part of the roadway, a concentrated load of 12 tons on two axles 10 feet centers and 5 feet gage (assumed to occupy a width of 12 feet), or on each street car track a concentrated load of 24 tons on two axles 10 feet centers; and on the remaining portion of the floor, including footwalks, a load of 100 pounds per square foot.

Loads for the trusses as per Table I.

Class C. For Highway and Light Interurban Traffic.—For the floor and its supports, on any part of the roadway, a concentrated load of 12 tons on two axles 10 feet centers and 5 feet gage (assumed to occupy a width of 12 feet), or on each street car track a concentrated load of 18 tons on two axles 10 feet centers; and upon the remaining portion of the floor, including footwalks, a load of 100 pounds per square foot.

Loads for the trusses as per Table I.

Class D_1. Heavy Country Bridges.—For the floor and its supports, a load of 100 pounds per square foot of total floor surface or a 12-ton traction engine with axles 10 feet centers and 6 feet gage, two-thirds of the load to be carried on the rear axles.

Loads for the trusses as per Table I. No bridge, however, to be designed for a load of less than 1,000 pounds per lineal foot of bridge.

Class D_2. Ordinary Country Bridges.—For the floor and its supports, a load of 80 pounds per square foot of total floor surface or an 8-ton traction engine with axles 8 feet centers and 6 feet gage, two-thirds of the load to be carried on the rear axles.

Loads for the trusses as per Table I. No bridge, however, to be designed for a load of less than 800 pounds per lineal foot of bridge.

Class E_1. For Heavy Electric Railways Only.—On each track a series of concentrations consisting of two pairs of trucks, the axles of the pairs being spaced 5 ft. centers, while the distance between centers of interior axles is 10 ft., the pairs of trucks being spaced 15 ft. centers. The axles are loaded with a

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load of 40,000 lbs., making a total of 160,000 lbs. Or a uniform load of 6,000 lbs. per lineal foot for all spans up to 50 ft., reduced to 4,500 lbs. per lineal foot for spans of 200 ft. and over, and proportionately for intermediate spans.

Class E_2. For Medium Electric Railways Only.—On each track a series of concentrations consisting of two pairs of trucks, the axles of the pairs being spaced 5 ft. centers, while the distance between centers of interior axles is 10 ft., the pairs of trucks being spaced 15 ft. centers. The axles are loaded with a load of 25,000 lbs., making a total load of 100,000 lbs. Or a uniform load of 3,500 lbs. per lineal foot for all spans up to 50 ft., reduced to 2,000 lbs. per lineal foot for spans of 200 ft. and over, and proportionately for intermediate spans.

Class E_3. For Light Electric Railways Only.—On each track a series of concentrations consisting of two pairs of trucks, the axles of the pairs being spaced 5 ft. centers, while the distance between centers of interior axles is 10 ft., the pairs of trucks being spaced 15 ft. centers. The axles are loaded with a load of 20,000 lbs., making a total load of 80,000 lbs. Or a uniform load of 2,500 lbs. per lineal foot for all spans up to 50 ft., reduced to 1,500 lbs. per lineal foot for spans of 200 ft. and over, and proportionately for intermediate spans.

TABLE I.

Live Loads for the Trusses.

Span in Feet.	Class A.		Class B.		Class C.		Class D_1.	Class D_2.
	Pounds per Lineal Foot of Each Car Track.	Pounds per Square Foot of Remaining Floor Surface.	Pounds per Lineal Foot of Each Car Track.	Pounds per Square Foot of Remaining Floor Surface.	Pounds per Lineal Foot of Each Car Track.	Pounds per Square Foot of Remaining Floor Surface.	Pounds per Square Foot of Floor Surface.	Pounds per Square Foot of Floor Surface.
Up to 100	1,800	100	1,800	80	1,200	80	80	75
105	1,770	99	1,770	79	1,190	79	79	74
110	1,740	98	1,740	78	1,180	78	78	73
115	1,710	97	1,710	77	1,170	77	77	72
120	1,680	96	1,680	76	1,160	76	76	71
125	1,650	95	1,650	75	1,150	75	75	70
130	1,620	94	1,620	74	1,140	74	74	69
135	1,590	93	1,590	73	1,130	73	73	68
140	1,560	92	1,560	72	1,120	72	72	67
145	1,530	91	1,530	71	1,110	71	71	66
150	1,500	90	1,500	70	1,100	70	70	65
155	1,470	89	1,470	69	1,090	69	69	64
160	1,440	88	1,440	68	1,080	68	68	63
165	1,410	87	1,410	67	1,070	67	67	62
170	1,380	86	1,380	66	1,060	66	66	61
175	1,350	85	1,350	65	1,050	65	65	60
180	1,320	84	1,320	64	1,040	64	64	59
185	1,290	83	1,290	63	1,030	63	63	58
190	1,260	82	1,260	62	1,020	62	62	57
195	1,230	81	1,230	61	1,010	61	61	56
200 and over	1,200	80	1,200	60	1,000	60	60	55

30. **Wind Loads.**—The top lateral bracing in deck bridges and the bottom lateral bracing in through bridges, shall be designed to resist a lateral wind load

of 300 pounds for each foot of span; 150 pounds of this to be treated as a moving load.

The bottom lateral bracing in deck bridges, and the top lateral bracing in through bridges, shall be designed to resist a lateral wind force of 150 pounds for each foot of span. In bridges with sway bracing one-half of the wind load may be assumed to pass to the lower chord through the sway bracing. For spans exceeding 300 feet, add in each of the above cases 10 pounds additional for each additional 30 feet.

31. In trestle towers the bracing and columns shall be designed to resist the following lateral forces, in addition to the stresses due to dead and live loads: The trusses loaded or unloaded the lateral pressures specified above; and a lateral pressure of 100 pounds for each vertical lineal foot of trestle bent.

32. **Temperature.**—Stresses due to a variation in temperature of 150 degrees shall be provided for (§ 81).

33. **Centrifugal Force of Train.**—Structures located on curves shall be designed for the centrifugal force of the live load acting at the top of the rail. The centrifugal force shall be calculated by the following formula:

$$C = (0.043 - 0.003D)W.D$$

where $C =$ centrifugal force in lbs.
$W =$ weight of train in lbs.
$D =$ degree of curvature.

34. **Longitudinal Forces.**—The stresses produced in the bracing of the trestle towers, in any members of the trusses, or in the attachments of the girders or trusses to their bearings, by suddenly stopping the maximum electric car trains on any part of the work must be provided for; the coefficient of friction of the wheels on the rails being assumed as 0.20.

35. All parts shall be so designed that the stresses coming upon them can be accurately calculated.

PART III. UNIT STRESSES AND PROPORTION OF PARTS.*

36. **Unit Stresses.**—All parts of the structure shall be proportioned so that the sum of the maximum stresses shall not exceed the following amounts in lbs. per sq. in., except as modified by § 45 and § 48.

Impact.—The dynamic increment of the live load stress shall be added to the maximum live load stresses and shall be determined by the formula

$$I = S \cdot 150/(L + 300)$$

where $I =$ impact increment to be added to the live load stresses;
$S =$ computed live load stress;
$L =$ loaded length of bridge in feet producing the maximum stress in the member.

* Sections 36a to 42a, inclusive, may be substituted for sections 36 to 42, inclusive.

Impact shall not be added to the stresses produced by longitudinal, centrifugal and lateral or wind forces.

37. **Tension.**—Axial tension on net section........................16,000

38. **Compression.**—Axial compression on gross section.......16,000 — $70 \cdot l/r$ where "l" is the length of member in inches and "r" is the least radius of gyration in inches.

No compression member, however, shall have a length exceeding 100 times its least radius of gyration for main members or 120 times for laterals for classes A, B, C, E_1, E_2 and E_3; or 125 times its least radius of gyration for main members or 150 times for laterals for classes D_1 and D_2.

39. **Bending.**—Bending: on extreme fibers of rolled shapes, built sections and girders; net section................................16,000
on extreme fibers of pins....................................24,000

40. **Shearing.**—Shearing: shop driven rivets and pins................12,000
field driven rivets and turned bolts............................10,000
plate girder webs; gross section 10,000

41. **Bearing.**—Bearing: shop driven rivets and pins..................24,000
field driven rivets and turned bolts............................20,000
granite masonry and Portland cement concrete................. 600
sandstone and limestone...................................... 400
expansion rollers; per linear inch............................ 600 d
where "d" is the diameter of the roller in inches.

42. **Alternate Stresses.**—Members subject to alternate stresses of tension and compression shall be proportioned for the stresses giving the largest section. If the alternate stresses occur in succession during the passage of one train, as in stiff counters, each stress shall be increased by 50 per cent of the smaller. The connections shall in all cases be proportioned for the sum of of the stresses.

(*Alternate Allowable Stresses.*—Sections 36a to 42a, inclusive, may be substituted for sections 36 to 42, inclusive.

36a. **Tensile Stresses.**—All parts of the structure shall be proportioned in tension by the following allowed unit stresses in lbs. per sq. in.:

Floorbeam hangers and other similar members liable to sudden loading, net section 8,000
Longitudinal, lateral and sway bracing, for wind and live load stresses.. 18,000
Solid rolled beams, used as cross floorbeams and stringers.............. 13,000
Bottom flanges of riveted girders, net section......................... 13,000
Bottom chords, main diagonals, counters and long verticals, 12,500 (for live loads), 25,000 (for dead loads).

For swing bridges and other movable structures, the dead load unit stresses, during motion, must not exceed three-fourths of the above allowed unit stresses for dead load on stationary structures.

37a. **Compresssive Stresses.**—Compression members shall be proportioned by the following allowed unit stresses:

Chord Segments

$P = 12{,}000 - 55 \cdot l/r$ for live load stresses

$P = 24{,}000 - 110 \cdot l/r$ for dead load stresses

All posts of through bridges

$$P = 10{,}000 - 45 \cdot l/r \text{ for live load stresses}$$
$$P = 20{,}000 - 90 \cdot l/r \text{ for dead load stresses}$$

All posts of deck bridges and trestles

$$P = 11{,}000 - 40 \cdot l/r \text{ for live load stresses}$$
$$P = 22{,}000 - 80 \cdot l/r \text{ for dead load stresses}$$

End-posts are not to be considered chord segments.

Lateral struts and rigid bracing

$$P = 13{,}000 - 60 \cdot l/r \text{ for wind stresses;}$$

for live load stresses use two-thirds of the above.

P = the allowed stress in compression per square inch of cross-section, in pounds; l = the length of compression member, in inches, c. to c., of connections; r = the least radius of gyration of the section, in inches.

No compression member, however, shall have a length exceeding 100 times its least radius of gyration for main members or 120 times for laterals for classes A, B, C, E_1, E_2 and E_3; or 125 times its least radius of gyration for main members, or 150 times for laterals for classes D_1 and D_2.

38a. **Bearing Stresses.**—Bearing on granite masonry and Portland cement concrete, 600 lbs. per sq. in.; bearing on sandstone and limestone, 400 lbs. per sq. in.; bearing on rollers = 400 lbs. per lineal inch, where d is the diameter of the roller in inches.

39a. **Bearing and Shear on Rivets.**—The rivets in all members, other than those of the floor and lateral systems, shall be so spaced that the shearing stress per square inch shall not exceed 10,000 pounds per square inch, nor the pressure on the bearing surface (diameter × thickness of the piece) of the rivet hole exceed 18,000 pounds per square inch.

The rivets in all members of the floor system, including all hanger connections, shall be so spaced that the shearing stresses and the bearing pressures shall not exceed 80 per cent of the above limits.

The rivets in the lateral systems will be allowed 40 per cent increase on the above limits.

In the case of field riveting and in the case of bolts the above allowed stresses will be reduced one-third.

Rivets and bolts shall not be used in direct tension.

40a. **Bearing, Shear, and Bending on Pins.**—Pins shall be proportioned so that the shearing stress shall not exceed 10,000 pounds per square inch; nor the pressure on the bearing surface of any member (other than forged eye-bars, § 76) connected to the pin be greater than 18,000 pounds per square inch; nor the bending stress exceed 20,000 pounds per square inch, when the applied forces are considered as acting at the centers of the bearing of each member.

41a. **Calculation of Areas.**—The areas obtained by dividing the live load stresses by the live load unit stresses shall be added to the areas obtained by dividing the dead load stresses by the dead load unit stresses to determine the sectional area of the member.

42a. **Alternate Stresses.**—All members and their connections subject to alternate tensile and compressive stresses shall be proportioned to resist each kind of stress. Both of the stresses shall, however, be considered as increased by $\frac{8}{10}$ of the least of the two stresses for determining the sectional areas by means of the above allowable unit stresses.)

43. **Angles Fastened by Both Legs.**—Angles subject to direct tension must be connected by both legs, or the section of one leg only will be considered as effective.

44. **Net Section.**—In members subject to tensile stresses full allowance shall be made for reduction of section by rivet-holes, screw-threads, etc.

45. **Long Span Bridges.**—For long span bridges, where the ratio of the length to width of span is such that it makes the top chords acting as a whole, a longer column than the segments of the chords, the chord shall be proportioned for the greater length.

46. **Wind Stresses.**—The stresses in truss members or trestle posts from assumed wind forces need not be considered except as follows:

1. When the wind stresses per square inch in any member exceeds 25 per cent of the stresses due to dead and live loads in the same member. The section shall then be increased until the total unit stress shall not exceed by more than 25 per cent the maximum allowable stress for dead and live loads.
2. When the wind stress alone or in combination with a possible temperature stress can neutralize or reverse the stresses in the member.

47. **Combined Stresses.**—Members subjected to direct and bending stresses shall be designed so that the greatest fiber stress shall not exceed the allowable unit stress on the member.

48. **Stress Due to Weight and Eccentric Loading.**—If the fiber stress due to weight and eccentric loading on any member exceeds 10 per cent of the allowable unit stress on the member such excess must be considered in proportioning the member.

49. **Counters.**—Counters in bridges carrying electric cars shall be designed so that an increase of the live load will not increase the stress in the counters more than 25 per cent.

50. **Design of Plate Girders.**—In designing plate girders the flanges shall be assumed to take all the bending moment and the web shall be assumed to take all the shear.

The distance between centers of gravity of flanges shall be considered as the effective depth.

Compression Flanges.—In beams and plate girders the compression flanges shall have the same gross section as the tension flanges.

51. **Web Plates.**—The webs of plate girders must be stiffened at intervals, not exceeding the depth of the girder or a maximum of 5 feet, wherever the shearing stress per square inch exceeds the stress allowed by the following formula:

$$\text{Allowed shearing stress} = 12{,}500 - 90H,$$

where H = ratio of depth of web to its thickness; but no web-plates shall be less than $\frac{5}{16}$ of an inch in thickness.

52. **Stiffeners.**—All stiffeners must be capable of carrying the maximum vertical shear without exceeding the allowed unit stress.

$$P = 12{,}000 - 55 \cdot l/r$$

Each stiffener must connect to the webs by enough rivets to transfer the maximum shear to or from the webs.

53. **Flange Rivets.**—The flanges of plate girders shall be connected to the web with a sufficient number of rivets to transfer the total shear at any point in a distance equal to the effective depth of the girder at that point combined with any load that is applied directly on the flange. The wheel loads, where the ties rest on the flanges, shall be assumed to be distributed over three ties.

54. **Depth Ratios.**—Trusses shall preferably have a depth of not less than one-tenth of the span. Plate girders and rolled beams, used as girders, shall preferably have a depth of not less than one-twelfth of the span. If shallower trusses, girders or beams are used, the section shall be increased so that the maximum deflection will not be greater than if the above limiting ratios had not been exceeded.

55. **Rolled Beams.**—Rolled beams shall be designed by using their moments of inertia.

PART IV. DETAILS OF DESIGN.

General Requirements.

56. **Open Sections.**—Structures shall be so designed that all parts will be accessible for inspection, cleaning and painting.

57. **Water Pockets.**—Pockets or depressions which would hold water shall have drain holes, or be filled with waterproof material.

58. **Symmetrical Sections.**—Main members shall be so designed that the neutral axis will be as nearly as practicable in the center of section, and the neutral axes of intersecting main members of trusses shall meet at a common point.

59. **Counters.**—Rigid counters are preferred; and where subject to reversal of stress shall preferably have riveted connections to the chords. Adjustable counters shall have open turnbuckles.

60. **Strength of Connections.**—The strength of connections shall be sufficient to develop the full strength of the member, even though the computed stress is less, the kind of stress to which the member is subjected being considered.

61. **Minimum Thickness.**—The minimum thickness of metal shall be $\frac{5}{16}$ in. in classes A, B, C, E_1, E_2 and E_3, except for fillers; and $\frac{1}{4}$ in. in classes D_1 and D_2, except for fillers. The minimum angle shall be $2'' \times 2'' \times \frac{1}{4}''$. The minimum rod shall have an area of at least 1 sq. in., in all classes except D_1 and D_2, which shall have no rods less than $\frac{3}{4}$ in. in diameter.

62. **Pitch of Rivets.**—The minimum distance between centers of rivet holes shall be three diameters of the rivet; but the distance shall preferably be not less than 3 in. for ⅞-in. rivets, 2½ in. for ¾-in. rivets, and 2 in. for ⅝-in. rivets. The maximum pitch in the line of stress for members composed of plates and shapes shall be 16 times the thickness of the thinnest outside plate or 6 in. For angles with two gage lines and rivets staggered, the maximum shall be twice the above in each line. Where two or more plates are used in contact, rivets not more than 12 in. apart in either direction shall be used to hold the plates well together. In tension members composed of two angles in contact, a pitch of 12 in. will be allowed for riveting the angles together.

63. **Edge Distance.**—The minimum distance from the center of any rivet hole to a sheared edge shall be 1½ in. for ⅞-in. rivets, 1¼ in. for ¾-in. rivets, and 1⅛ in. for ⅝-in. rivets; and to a rolled edge 1¼, 1⅛ and 1 in., respectively. The maximum distance from any edge shall be eight times the thickness of the plate, but shall not exceed 6 in.

64. **Maximum Diameter.**—The diameter of the rivets in any angle carrying calculated stress shall not exceed one-quarter the width of the leg in which they are driven. In minor parts ⅞-in. rivets may be used in 3-in. angles, ¾-in. rivets in 2½-in. angles, and ⅝-in. rivets in 2-in. angles.

65. **Long Rivets.**—Rivets carrying calculated stress and whose grip exceeds four diameters shall be increased in number at least one per cent for each additional 1/16-in. of grip.

66. **Pitch at Ends.**—The pitch of rivets at the ends of built compression members shall not exceed four diameters of the rivets, for a length equal to one and one-half times the maximum width of member.

67. **Compression Members.**—In compression members the metal shall be concentrated as much as possible in webs and flanges. The thickness of each web shall be not less than one-thirtieth of the distance between its connections to the flanges. Cover plates shall have a thickness not less than one-fortieth of the distance between rivet lines.

68. **Minimum Angles.**—Flanges of girders and built members without cover plates shall have a mimimum thickness of one-twelfth of the width of the outstanding leg.

69. **Batten Plates.**—The open sides of all compression members shall be stayed by batten plates at the ends and diagonal lattice-work at intermediate points. The batten plates must be placed as near the ends as practicable, and shall have a length not less than the greatest width of the member or 1½ times its least width.

70. **Lattice Bars.**—The size and spacing of the lattice bars shall be duly proportioned to the size of the member. They must not be less in width than 1½ inches for members 6 inches in width, 1¾ inches for members 9 inches in width, 2 inches for members 12 inches in width, 2¼ inches for members 15 inches in width, nor 2½ inches for members 18 inches and over in width. Single lattice bars shall have a thickness not less than one-fortieth, or double lattice bars connected by a rivet at the intersection, not less than one-sixtieth of the distance

between the rivets connecting them to the members. They shall be inclined at an angle not less than 60° to the axis of the member for single latticing, nor less than 45° for double latticing with riveted intersections.

71. **Spacing of Lattice Bars.**—Lattice bars shall be so spaced that the portion of the flange included between their connection shall be as strong as the member as a whole. The pitch of the lattice bars must not exceed the width of the channel plus nine inches.

72. **Rivets in Flanges.**—Five-eighths-inch rivets shall be used for latticing flanges less than $2\frac{1}{2}$ in. wide; $\frac{3}{4}$-in. for flanges from $2\frac{1}{2}$ to $3\frac{1}{2}$ in. wide; $\frac{7}{8}$-in. rivets shall be used in flanges $3\frac{1}{2}$ in. and over, and lattice bars with two rivets shall be used for flanges over 5 in. wide.

73. **Faced Joints.**—Abutting joints in compression members, when faced for bearing, shall be spliced on four sides sufficiently to hold the connecting members accurately in place. All other joints in riveted work, whether in tension or compression, shall be fully spliced.

74. **Pin Plates.**—Where necessary, pin-holes shall be reinforced by plates, some of which must be of the full width of the member, so the allowed pressure on the pins shall not be exceeded, and so the stresses shall be properly distributed over the full cross-section of the members. These reinforcing plates must contain enough rivets to transfer their proportion of the bearing pressure, and at least one plate on each side shall extend not less than 6 inches beyond the edge of the nearest batten plate.

75. **Riveted Tension Members.**—Riveted tension members shall have an effective section through the pin-holes 25 per cent in excess of the net section of the member, and back of the pin at least 75 per cent of the net section through the pin-hole.

76. **Pins.**—Pins shall be long enough to insure a full bearing of all the parts connected upon the turned body of the pin. The diameter of the pin shall not be less than $\frac{3}{4}$ of the thickness of any eye-bar attached to it. They shall be secured by chambered Lomas nuts or be provided with washers if solid nuts are used. The screw ends shall be long enough to admit of burring the threads.

77. **Filling Rings.**—Members packed on pins shall be held against lateral movement.

78. **Bolts.**—Where members are connected by bolts, the turned body of these bolts shall be long enough to extend through the metal. A washer at least $\frac{1}{4}$ in. thick shall be used under the nut. Bolts shall not be used in place of rivets except by special permission. Heads and nuts shall be hexagonal.

79. **Indirect Splices.**—Where splice plates are not in direct contact with the parts which they connect, rivets shall be used on each side of the joint in excess of the number theoretically required to the extent of one-third of the number for each intervening plate.

80. **Fillers.**—Rivets carrying stress and passing through fillers shall be increased 50 per cent in number; and the excess rivets, when possible, shall be outside of the connected member.

81. **Expansion.**—Provision for expansion to the extent of $\frac{1}{8}$ in. for each 10 ft. shall be made for all bridge structures. Efficient means shall be provided to prevent excessive motion at any one point (§ 32).

82. **Expansion Bearings.**—Spans of 80 ft. and over resting on masonry shall have turned rollers or rockers at one end; and those of less length shall be arranged to slide on smooth surfaces.

83. **Fixed Bearings.**—Movable bearings shall be designed to permit motion in one direction only. Fixed bearings shall be firmly anchored to the masonry (§ 87).

84. **Rollers.**—Expansion rollers shall be not less than 3 in. in diameter for spans of 100 feet and less, and shall be increased 1 inch for each 100 feet additional. They shall be coupled together with substantial side bars, which shall be so arranged that the rollers can be readily cleaned.

85. **Bolsters.**—Bolsters or shoes shall be so constructed that the load will be distributed over the entire bearing.

86. **Pedestals and Bed-Plates.**—Pedestals shall be made of riveted plates and angles. All bearing surfaces of the base plates and vertical webs must be planed. The vertical webs must be secured to the base by angles having two rows of rivets in the vertical legs. No base plate or web connecting angle shall be less in thickness than $\frac{1}{2}$ inch. The vertical webs shall be of sufficient height and must contain material and rivets enough to practically distribute the loads over the bearings or rollers.

Where the size of the pedestal permits, the vertical webs must be rigidly connected transversely.

87. All the bed-plates and bearings under fixed and movable ends must be fox-bolted to the masonry; for trusses, these bolts must not be less than $1\frac{1}{4}$ inches diameter; for plate and other girders, not less than $\frac{7}{8}$ inch diameter.

88. **Wall Plates.**—Wall plates may be cast or built up; and shall be so designed as to distribute the load uniformly over the entire bearing. They shall be secured against displacement.

89. **Anchorage.**—Anchor bolts for viaduct towers and similar structures shall be long enough to engage a mass of masonry the weight of which is at least one and one-half times the uplift (§ 11).

90. **Inclined Bearings.**—Bridges on an inclined grade without pin shoes shall have the sole plates beveled so that the masonry and expansion surfaces may be level.

91. **Camber.**—Truss spans shall be given a camber by making the panel length of the top chords, or their horizontal projections, longer than the corresponding panels of the bottom chord in the proportion of $\frac{3}{16}$ in. in 10 ft.

92. **Eye-bars.**—The eye-bars composing a member shall be so arranged that adjacent bars shall not have their surfaces in contact; they shall be as nearly parallel to the axis of the truss as possible, the maximum inclination of any bar being limited to one inch in 16 ft.

PART V. MATERIALS AND WORKMANSHIP.

MATERIAL.

93. **Process of Manufacture.**—Steel shall be made by the open-hearth process.

94. **Schedule of Requirements.**—The chemical and physical properties shall conform to the following limits:

ELEMENTS CONSIDERED.	STRUCTURAL STEEL.	RIVET STEEL.	STEEL CASTINGS.
Phosphorus, max........ { Basic......	0.04 per cent.	0.04 per cent.	0.05 per cent.
Phosphorus, max........ { Acid	0.06 " "	0.04 " "	0.08 " "
Sulphur, maximum....................	0.05 " "	0.04 " "	0.05 " "
Ultimate tensile strength Pounds, per square inch..............	Desired 60,000	Desired 50,000	Not less than 65,000
Elong., min. % in 8″, Fig. 1...	$\frac{1,500,000}{\text{Ult. tensile str'gth.}}$ *	$\frac{1,500,000}{\text{Ult. tensile str'gth.}}$	15 per cent.
" " " 2″, " 2......	22		
Character of Fracture..................	Silky	Silky	Silky or fine granular
Cold Bends without Fracture.........	180° flat †	180° flat ‡	90° $d = 3t$.

The yield point, as indicated by the drop of beam, shall be recorded in the test reports.

95. **Allowable Variations.**—If the ultimate strength varies more than 4,000 lbs. from that desired, a retest shall be made on the same gage, which, to be acceptable, shall be within 5,000 lbs. of the desired ultimate.

96. **Chemical Analyses.**—Chemical determinations of the percentages of carbon, phosphorus, sulphur and manganese shall be made by the manufacturer from a test ingot taken at the time of the pouring of each melt of steel, and a correct copy of such analysis shall be furnished to the engineer or his inspector. Check analyses shall be made from finished material, if called for by the purchaser, in which case an excess of 25 per cent above the required limits will be allowed.

97. **Form of Specimens.** *Plates, Shapes and Bars.*—Specimens for tensile and bending tests for plates, shapes and bars shall be made by cutting coupons from the finished product, which shall have both faces rolled and both edges milled to the form shown by Fig. 1; or with both edges parallel; or they may be turned to a diameter of ¾ in. for a length of at least 9 in., with enlarged ends.

98. *Rivets.*—Rivet rods shall be tested as rolled.

99. *Pins and Rollers.*—Specimens shall be cut from the finished rolled or forged bar, in such manner that the center of the specimen shall be one inch from the surface of the bar. The specimen for tensile test shall be turned to

* See paragraph 103.

† See paragraphs 104, 105, and 106.

‡ See paragraph 107.

the form shown by Fig. 2. The specimen for bending test shall be one inch by one-half inch in section.

100. *Steel Castings.*—The number of tests will depend on the character and importance of the castings. Specimens shall be cut cold from coupons molded and cast on some portion of one or more castings from each melt or from the

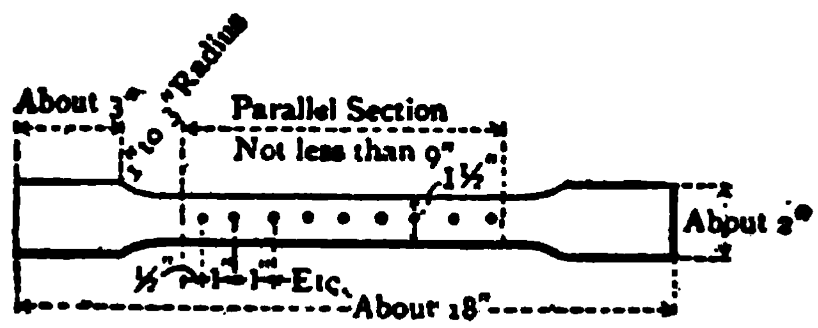

FIG. 1

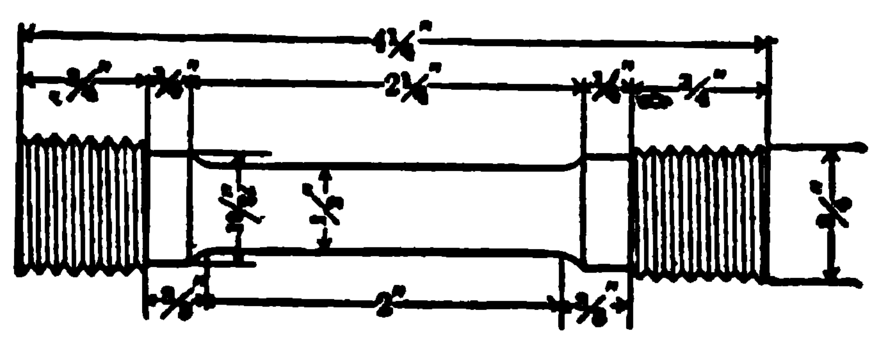

FIG. 2.

sink heads, if the heads are of sufficient size. The coupon or sink head, so used, shall be annealed with the casting before it is cut off. Test specimens to be of the form prescribed for pins and rollers.

101. **Annealed Specimens.**—Material which is to be used without annealing or further treatment shall be tested in the condition in which it comes from the rolls. When material is to be annealed, or otherwise treated before use, the specimens for tensile tests representing such material shall be cut from properly annealed or similarly treated short lengths of the full section of the bar.

102. **Number of Tests.**—At least one tensile and one bending test shall be made from each melt of steel as rolled. In case steel differing $\frac{3}{8}$ in. and more in thickness is rolled from one melt, a test shall be made from the thickest and thinnest material rolled.

103. **Modifications in Elongation.**—For material less than $\frac{5}{16}$ in. and more than $\frac{3}{4}$ in. in thickness the following modifications will be allowed in the requirements for elongation:

(*a*) For each $\frac{1}{16}$ in. in thickness below $\frac{5}{16}$ in., a deduction of $2\frac{1}{2}$ will be allowed from the specified percentage.

(*b*) For each $\frac{1}{8}$ in. in thickness above $\frac{3}{4}$ in., a deduction of 1 will be allowed from the specified percentage.

104. **Bending Tests.**—Bending tests may be made by pressure or by blows. Plates, shapes and bars less than one inch thick shall bend as called for in paragraph 94.

105. **Thick Material.**—Full-sized material for eye-bars and other steel one inch thick and over, tested as rolled, shall bend cold 180 degrees around a pin, the diameter of which is equal to twice the thickness of the bar, without fracture on the outside of bend.

106. **Bending Angles.**—Angles ¾ in. and less in thickness shall open flat, and angles ½ in. and less in thickness shall bend shut, cold, under blows of a hammer, without sign of fracture. This test will be made only when required by the inspector.

107. **Nicked Bends.**—Rivet steel, when nicked and bent around a bar of the same diameter as the rivet rod, shall give a gradual break and a fine, silky uniform fracture.

108. **Finish.**—Finished material shall be free from injurious seams, flaws, cracks, defective edges or other defects, and have a smooth, uniform and workmanlike finish. Plates 36 in. in width and under shall have rolled edges.

109. **Stamping.**—Every finished piece of steel shall have the melt number and the name of the manufacturer stamped or rolled upon it. Steel for pins and rollers shall be stamped on the end. Rivet and lattice steel and other small parts may be bundled with the above marks on an attached metal tag.

110. **Defective Material.**—Material which, subsequent to the above tests at the mills, and its acceptance there, develops weak spots, brittleness, cracks or other imperfections, or is found to have injurious defects, will be rejected at the shop and shall be replaced by the manufacturer at his own cost.

111. **Allowable Variation in Weight.**—A variation in cross-section or weight of each piece of steel of more than 2½ per cent from that specified will be sufficient cause for rejection, except in case of sheared plates, which will be covered by the following permissible variations, which are to apply to single plates:

112. **When Ordered to Weight.**—Plates 12½ lbs. per sq. ft. or heavier:

(*a*) Up to 100 in. wide 2½ per cent above or below the prescribed weight.

(*b*) One hundred inches wide and over, 5 per cent above or below.

113. Plates under 12½ lbs. per sq. ft.:

(*a*) Up to 75 in. wide, 2½ per cent above or below.

(*b*) Seventy-five inches and up to 100 in. wide, 5 per cent above or 3 per cent below.

(*c*) One hundred inches wide and over, 10 per cent above or 3 per cent below.

114. Plates will be accepted if they measure not more than 0.01 in. below the ordered thickness.

TABLE 1.

Thickness Ordered.	Nominal Weights	Width of Plate.			
		Up to 75″	75″ and up to 100″	100″ and up to 115″	Over 115″
¼ inch	10.20 lbs.	10 per cent.	14 per cent.	18 per cent.	....
$\frac{5}{16}$ "	12.75 "	8 "	12 "	16 "	....
⅜ "	15.30 "	7 "	10 "	13 "	17 per cent.
$\frac{7}{16}$ "	17.85 "	6 "	8 "	10 "	13 "
½ "	20.40 "	5 "	7 "	9 "	12 "
$\frac{9}{16}$ "	22.95 "	4½ "	6½ "	8½ "	11 "
⅝ "	25.50 "	4 "	6 "	8 "	10 "
Over ⅝ "	....	3½ "	5 "	6⅓ "	9 "

115. **When Ordered to Gage.**—An excess over the nominal weight, corresponding to the dimensions on the order, will be allowed for each plate, if not more than that shown in Table 1, one cu. in. of rolled steel being assumed to weigh 0.2833 lb.:

116. **Cast Iron.**—Except where chilled iron is specified, castings shall be made of tough gray iron, with sulphur not over 0.10 per cent. They shall be true to pattern, out of wind and free from flaws and excessive shrinkage. If tests are demanded, they shall be made on the "Arbitration Bar" of the American Society for Testing Materials, which is a round bar 1¼ in. diameter and 15 in. long. The transverse test shall be made on a supported length of 12 in. with load at middle. The minimum breaking load so applied shall be 2,900 lbs., with a deflection of at least $\frac{1}{10}$ in. before rupture.

117. **Wrought Iron Bars.**—Wrought iron shall be double-rolled, tough, fibrous and uniform in character. It shall be thoroughly welded in rolling and be free from surface defects. When tested in specimens of the form of Fig. 1, or in full-sized pieces of the same length, it shall show an ultimate strength of at least 50,000 lbs. per sq. in., an elongation of at least 18 per cent in 8 in., with fracture wholly fibrous. Specimens shall bend cold, with the fiber, through 135 degrees, without sign of fracture, around a pin the diameter of which is not over twice the thickness of the piece tested. When nicked and bent, the fracture shall show at least 90 per cent fibrous.

118. **Timber.**—The timber shall be strictly first-class spruce, white pine, Douglas fir, Southern yellow pine, or white oak bridge timber; sawed true and out of wind, full size, free from wind shakes, large or loose knots, decayed or sapwood, wormholes or other defects impairing its strength or durability.

Workmanship.

119. **General.**—All parts forming a structure shall be built in accordance with approved drawings. The workmanship and finish shall be equal to the best practice in modern bridge works.

120. **Straightening Material.**—Material shall be thoroughly straightened in the shop, by methods that will not injure it, before being laid off or worked in any way.

121. **Finish.**—Shearing shall be neatly and accurately done and all portions of the work exposed to view neatly finished.

122. **Size of Rivets.**—The size of rivets, called for on the plans, shall be understood to mean the actual size of the cold rivet before heating.

123. **Rivet Holes.**—When general reaming is not required the diameter of the punch shall not be more than $\frac{1}{16}$ in. greater than the diameter of the rivet; nor the diameter of the die more than ⅛ in. greater than the diameter of the punch. Material more than ¾ in. thick shall be sub-punched and reamed or drilled from the solid.

124. **Punching.**—All punching shall be accurately done. Drifting to enlarge unfair holes will not be allowed. If the holes must be enlarged to admit the rivet, they shall be reamed. Poor matching of holes will be cause for rejection.

125. **Sub-punching and Reaming.**—Where reaming is required, the punch used shall have a diameter not less than $\frac{3}{16}$ in. smaller than the nominal diameter of the rivet. Holes shall then be reamed to a diameter not more than $\frac{1}{16}$ in. larger than the nominal diameter of the rivet. All reaming small be done with twist drills. (§ 140.)

126. **Reaming After Assembling.**—When general reaming is required it shall be done after the pieces forming one built member are assembled and firmly bolted together. If necessary to take the pieces apart for shipping and handling, the respective pieces reamed together shall be so marked that they may be reassembled in the same position in the final setting up. No interchange of reamed parts will be allowed.

127. **Edge Planing.**—Sheared edges or ends shall, when required, be planed at least $\frac{1}{8}$ in.

128. **Burrs.**—The outside burrs on reamed holes shall be removed.

129. **Assembling.**—Riveted members shall have all parts well pinned up and firmly drawn together with bolts, before riveting is commenced. Contact surfaces to be painted.

130. **Lattice Bars.**—Lattice bars shall have neatly rounded ends, unless otherwise called for.

131. **Web Stiffeners.**—Stiffeners shall fit neatly between flanges of girders. Where tight fits are called for, the ends of the stiffeners shall be faced and shall be brought to a true contact bearing with the flange angles.

132. **Splice Plates and Fillers.**—Web splice plates and fillers under stiffeners shall be cut to fit within $\frac{1}{8}$ in. of flange angles.

133. **Web Plates.**—Web plates of girders, which have no cover plates, shall be flush with the backs of angles or project above the same not more than $\frac{1}{8}$ in., unless otherwise called for. When web plates are spliced, not more than $\frac{1}{4}$ in. clearance between ends of plates will be allowed.

134. **Connection Angles.**—Connection angles for floorbeams and stringers shall be flush with each other and correct as to position and length of girder. In case milling (of all such angles) is needed or is required after riveting, the removal of more than $\frac{1}{16}$ in. from their thickness will be cause for rejection.

135. **Rivets.**—Rivets shall be driven by pressure tools wherever possible. Pneumatic hammers shall be used in preference to hand driving.

136. **Riveting.**—Rivets shall look neat and finished, with heads of approved shape, full and of equal size. They shall be central on shank and grip the assembled pieces firmly. Recupping and calking will not be allowed. Loose, burned or otherwise defective rivets shall be cut out and replaced. In cutting out rivets, great care shall be taken not to injure the adjacent metal. If necessary, they shall be drilled out.

137. **Turned Bolts.**—Wherever bolts are used in place of rivets which transmit shear, the holes shall be reamed parallel and the bolts turned to a driving fit. A washer not less than $\frac{1}{4}$ in. thick shall be used under nut.

138. **Members to be Straight.**—The several pieces forming one built member shall be straight and fit closely together, and finished members shall be free from twists, bends or open joints.

139. **Finish of Joints.**—Abutting joints shall be cut or dressed true and straight and fitted close together, especially where open to view. In compression joints, depending on contact bearing, the surfaces shall be truly faced, so as to have even bearings after they are riveted up complete and when perfectly aligned.

140. **Field Connections.**—Holes for floorbeam and stringer connections shall be sub-punched and reamed according to paragraph 125, to a steel templet one inch thick. (If required, all other field connections, except those for laterals and sway bracing, shall be assembled in the shop and the unfair holes reamed; and when so reamed, the pieces shall be match-marked before being taken apart.)

141. **Eye-Bars.**—Eye-bars shall be straight and true to size, and shall be free from twists, folds in the neck or head, or any other defect. Heads shall be made by upsetting, rolling or forging. Welding will not be allowed. The form of heads will be determined by the dies in use at the works where the eye-bars are made, if satisfactory to the engineer, but the manufacturer shall guarantee the bars to break in the body when tested to rupture. The thickness of head and neck shall not vary more than $\frac{1}{16}$ in. from that specified.

142. **Boring Eye-Bars.**—Before boring, each eye-bar shall be properly annealed and carefully straightened. Pin-holes shall be in the center line of bars and in the center of heads. Bars of the same length shall be bored so accurately that, when placed together, pins $\frac{1}{32}$ in. smaller in diameter than the pin-holes can be passed through the holes at both ends of the bars at the same time without forcing.

143. **Pin-Holes.**—Pin-holes shall be bored true to gages, smooth and straight; at right angles to the axis of the member and parallel to each other, unless otherwise called for. The boring shall be done after the member is riveted up.

144. **Variation in Pin-Holes.**—The distance center to center of pin-holes shall be correct within $\frac{1}{32}$ in., and the diameter of the holes not more than $\frac{1}{50}$ in. larger than that of the pin, for pins up to 5-in. diameter, and $\frac{1}{32}$ in. for larger pins.

145. **Pins and Rollers.**—Pins and rollers shall be accurately turned to gages and shall be straight and smooth and entirely free from flaws.

146. **Screw Threads.**—Screw threads shall make tight fits in the nuts and shall be U. S. standard, except above the diameter of $1\frac{3}{8}$ in., when they shall be made with six threads per inch.

147. **Annealing.**—Steel, except in minor details, which has been partially heated, shall be properly annealed.

148. **Steel Castings.**—All steel castings shall be annealed.

149. **Welds.**—Welds in steel will not be allowed.

150. **Bed Plates.**—Expansion bed plates shall be planed true and smooth. Cast wall plates shall be planed top and bottom. The cut of the planing tool shall correspond with the direction of expansion.

151. **Pilot Nuts.**—Pilot and driving nuts shall be furnished for each size of pin, in such numbers as may be ordered.

152. **Field Rivets.**—Field rivets shall be furnished to the amount of 15 per cent plus ten rivets in excess of the nominal number required for each size.

153. **Shipping Details.**—Pins, nuts, bolts, rivets and other small details shall be boxed or crated.

154. **Weight.**—The weight of every piece and box shall be marked on it in plain figures.

155. **Finished Weight.**—Payment for pound price contracts shall be by scale weight. No allowance over 2 per cent of the total weight of the structure as computed from the plans will be allowed for excess weight.

SHOP PAINTING.

156. **Cleaning.**—Steel work, before leaving the shop, shall be thoroughly cleaned and given one good coating of pure linseed oil, or such paint as may be called for, well worked into all joints and open spaces.

157. **Contact Surfaces.**—In riveted work, the surfaces coming in contact shall each be painted before being riveted together.

158. **Inaccessible Surfaces.**—Pieces and parts which are not accessible for painting after erection, including tops of stringers, eye-bar heads, ends of posts and chords, etc., shall have a good coat of paint before leaving the shop.

159. **Condition of Surfaces.**—Painting shall be done only when the surface of the metal is perfectly dry. It shall not be done in wet or freezing weather, unless protected under cover.

160. **Machine-finished Surfaces.**—Machine-finished surfaces shall be coated with white lead and tallow before shipment or before being put out into the open air.

INSPECTION AND TESTING AT THE SHOP AND MILL.

161. **Facilities for Shop Inspection.**—The manufacturer shall furnish all facilities for inspecting and testing the weight and quality of workmanship at the shop where material is manufactured. He shall furnish a suitable testing machine for testing full-sized members, if required.

162. **Starting Work in Shop.**—The purchaser shall be notified well in advance of the start of the work in the shop, in order that he may have an inspector on hand to inspect material and workmanship.

163. **Copies of Mill Orders.**—The purchaser shall be furnished complete copies of mill orders, and no material shall be rolled, nor work done, before the purchaser has been notified where the orders have been placed, so that he may arrange for the inspection.

164. **Facilities for Mill Inspection.**—The manufacturer shall furnish all facilities for inspecting and testing the weight and quality of all material at the mill where it is manufactured. He shall furnish a suitable testing machine for testing the specimens, as well as prepare the pieces for the machine, free of cost.

165. **Access to Mills.**—When an inspector is furnished by the purchaser to inspect material at the mills, he shall have full access, at all times, to all parts of mills where material to be inspected by him is being manufactured.

166. **Access to Shop.**—When an inspector is furnished by the purchaser, he shall have full access, at all times, to all parts of the shop where material under his inspection is being manufactured.

167. **Accepting Material or Work.**—The inspector shall stamp each piece accepted with a private mark. Any piece not so marked may be rejected at any time, and at any stage of the work. If the inspector, through an oversight or otherwise, has accepted material or work which is defective or contrary to the specifications, this material, no matter in what stage of completion, may be rejected by the purchaser.

168. **Shop Plans.**—The purchaser shall be furnished complete shop plans (§ 13).

169. **Shipping Invoices.**—Complete copies of shipping invoices shall be furnished to the purchaser with each shipment.

FULL-SIZED TESTS.

170. **Test to Prove Workmanship.**—Full-sized tests on eye-bars and similar members, to prove the workmanship, shall be made at the manufacturer's expense, and shall be paid for by the purchaser at contract price, if the tests are satisfactory. If the tests are not satisfactory, the members represented by them will be rejected.

171. **Eye-bar Tests.**—In eye-bar tests, the fracture shall be silky, the elongation in 10 ft., including the fracture, shall be not less than 15 per cent; and the ultimate strength and true elastic limit shall be recorded. (§ 141.)

ERECTION.

172. If the contractor erects the bridge he shall, unless otherwise specified, furnish all staging and falsework, erect and adjust all metal work, and shall frame and put in place all floor timbers, guard timbers, trestle timbers, etc., complete ready for traffic.

173. The contractor shall put in place all stone bolts and anchors for attaching the steel work to the masonry. He shall drill all the necessary holes in the masonry, and set all bolts with neat Portland cement.

174. The erection will also include all necessary hauling from the railroad station, the unloading of the materials and their proper care until the erection is completed.

175. Whenever new structures are to replace existing ones, the latter are to be carefully taken down and removed by the contractor to some place where the material can be hauled away.

176. The contractor shall so conduct his work as not to interfere with traffic, interfere with the work of other contractors, or close any thoroughfare on land or water.

177. The contractor shall assume all risks of accidents and damages to persons and properties prior to the acceptance of the work.

178. The contractor must remove all falsework, piling and other obstructions or unsightly material produced by his operations.

PAINTING AFTER ERECTION.

179. After the bridge is erected the metal work shall be thoroughly cleaned of mud, grease or other material, then thoroughly and evenly painted with two coats of paint of the kind specified by the engineer, mixed with linseed oil. All recesses which may retain water, or through which water can enter, must be filled with thick paint or some waterproof cement before the final painting. The different coats of paint must be of distinctly different shades or colors, and one coat must be allowed to dry thoroughly before the second coat is applied. No painting shall be done in wet or freezing weather.

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